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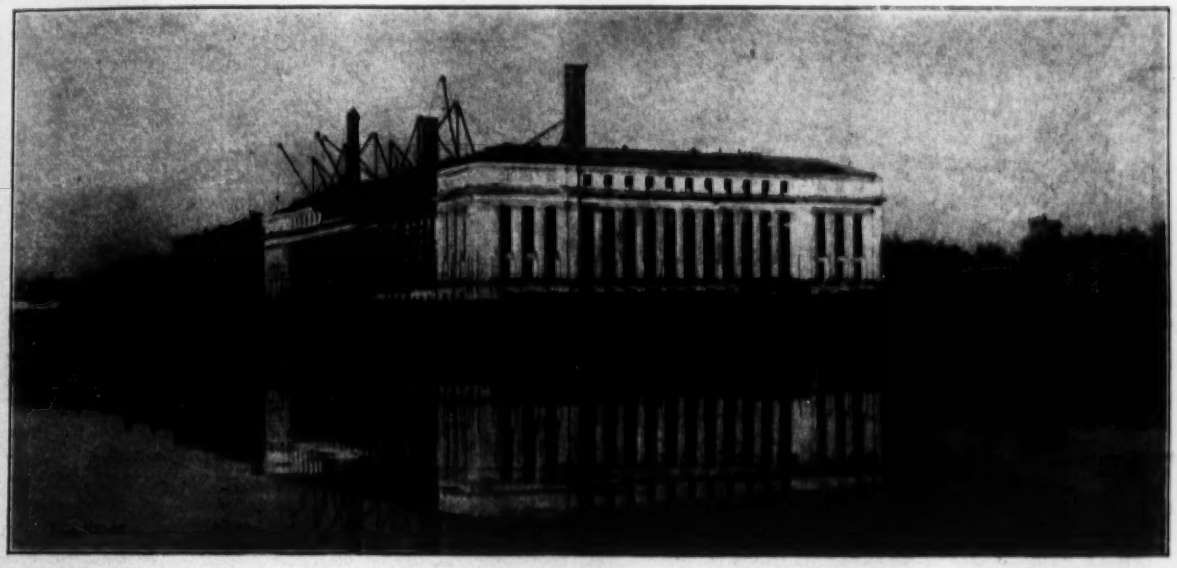
The New Building for the Bureau of Engraving and Printing

By Fred W. Lepper*

During the construction of a public building, the question most frequently asked is: "Why should it take the Government so much longer to do a piece of work than it takes anyone else to do it"? That the implication is not true in all cases is proved by the manner in which the construction of the new building for the Bureau of Engraving and Printing at Washington has recently been carried on.



The New Bureau of Engraving and Printing Building, Washington, D. C.

The new structure, a modern high-class factory building, is 505 ft. long by 300 ft. deep, with three courts 85x228 ft. in the rear; basement and five stories in height; more than half a mile around the outside walls; and having about eleven acres of total floor area. It has a granite base and heavy limestone facing on all walls, steel frame and reinforced-concrete floors, and is strictly fireproof throughout, with metal window-frames and sash. Nearly two-thirds of the wall area, more exactly 60%, is of glass. For the comfort of the employees during the summer months, recreation promenades with awning covers will be provided on the roofs of the two center wings. The inside finish of the walls is generally a 5-ft. wainscot of red face-brick and white sand-lime brick above. The cost of the building complete, including heating and ventilating equipment, wiring, elevators and vaults will be $2,325,000, or about 25c. per cu.ft.

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Progress Schedule

Beginning in the dead of the severe winter of 1911-12 the excavation of 74,000 cu.yd. was done and the concrete footings were placed, involving 6000 cu.yd. This part of the work was done by the middle of May, 1912. The erection of the 4000 tons of steel framing was commenced about this time. The frame was erected at a rate which permitted the placing of 10,000 cu.yd. of reinforced concrete and the setting of 250,000 cu.ft. of limestone on the walls in about six months, that is, by the end of December, 1912.

The accompanying diagram, Fig. 2, illustrates graphically the rates of progress made. The rates anticipated before the work was commenced and the actually realized rates are shown.

Erection Plant and Methods

Two steam shovels were used on the excavation, the material from which was delivered by teams to barges and towed to the river flats for filling.

Three batch mixers were used on the concrete foundation work, which is made up of simple spread and stepped footings. The type of forms for the latter, especially those for the isolated pier work, shown at the left in Fig. 5, were found to be most effective. They were made up generally of 2x12-in. planks halved to-

gether at the corners and wedged, without the use of bolts or spikes of any kind.

The layout of the erection plant is shown in Fig. 4, a general plan of the building and site, giving the principal dimensions and showing the arrangement of 16 stiff-leg derricks, temporary tracks and other equipment for handling the materials. Fig. 1, a progress photograph, shows these derricks set up on the fourth floor. The derricks were of 5 and 10 tons capacity and nearly all were operated by 30- and 40-hp. electric hoists, often more than one from the same hoist. In order that there might be no delay in the commencement of the stonework, the derricks were set up on the fourth-floor framing and later raised to the roof. A 15-ton locomotive crane was kept constantly employed and proved a most effective and economical part of the equipment in the handling of materials in carload lots.

The roof construction is of 3-in. slabs on steel bea... All steel girders and beams are fireproofed with ... crete. The total amount of concrete in the floor ... roof construction is something more than 10,000 cu...

All this concrete was handled by the duplicate pla... shown at the right in Fig. 4. The sand and gravel w... dumped through a grating into the pit *A*, lifted to ... 60-yd. storage bin *B* by a 5-hp. bucket conveyor, me... ured in the divided hopper *C*, where the cement w... added and discharged into the batch mixer *D*. The co... crete was dumped into the ¾-yd. bucket *E*, hoisted to t... necessary height by a 10-hp. motor, and automatically dumped into the hopper *F*, where a valve controlled ta... flow of the concrete into the chute *G* to the hopper *H*, from which it was distributed by secondary chutes or by barrows to the desired point. Only electric power was used in these plants.

FIG. 1. ERECTION OF NEW BUILDING FOR BUREAU OF ENGRAVING AND PRINTING, AUG. 1, 1912

The general framing of the building, indicated in part by Fig. 3, is of plate-and-angle columns in two-story lengths, with I-beams as girders or main beams on the lines of column centers. The girders in the longer spans are specially-rolled 120-lb. beams 27-in. deep, having a section modulus of 229³. The riveting of the steelwork was done by six guns. These were supplied with compressed air from a fixed compressor plant of about 40 hp. placed in the basement at the center of the west section of the building, from which point all air piping was run.

The floor construction (see sketch section in Fig. 3) is of 1:2:4 reinforced gravel-concrete, generally 3-in. slabs, reinforced with rib metal, on reinforced-concrete intermediate beams 12½ in. deep, sloping from 18½ in. top width to 11½ in. bottom width. These beams are reinforced with Kahn bars with additional shear bars.

The concrete was mixed just wet enough to prevent its clogging the chutes. These were lined with sheet metal, and their slope was limited to 45°. The towers were about 150 ft. in height, guyed at two points with ⅝-in. wire rope; they were constructed of members of the sizes marked on Fig. 6.

The capacity of each plant was about 170 cu.yd. in 8 hr. The bins were an especially effective part of the plant, by their storage of concrete materials, thereby saving many short delays from irregular deliveries.

As the intention is to have no plaster finish on the ceilings, an essential requirement in the concrete work was that it should be unusually smooth on the under side and should not be patched. This result was attained by using wood forms of ⅞-in. t. & g. material painted with paraffin oil, well stiffened and shored every three feet with 4x4-in. posts; no hangers were al-

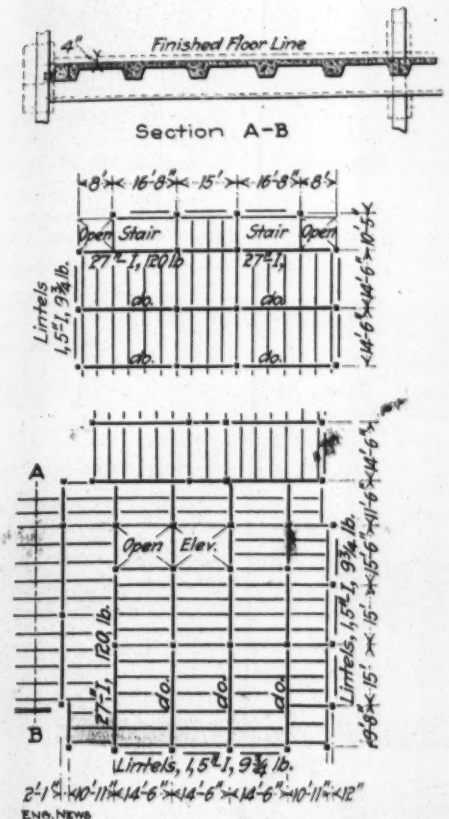


FIG. 3. PART FRAMING PLAN THIRD FLOOR, SHOWING TYPICAL FLOOR CONSTRUCTION

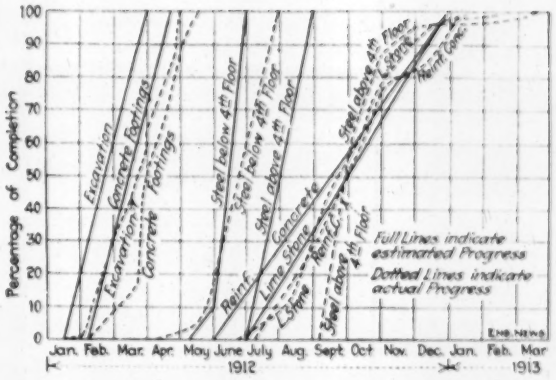
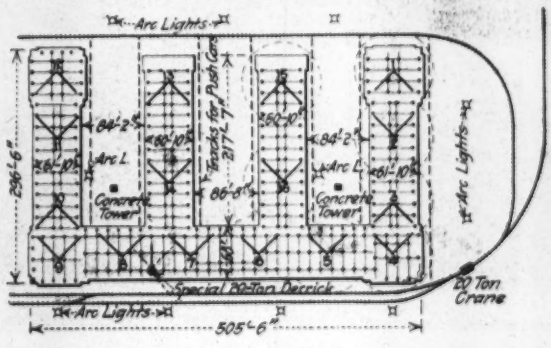
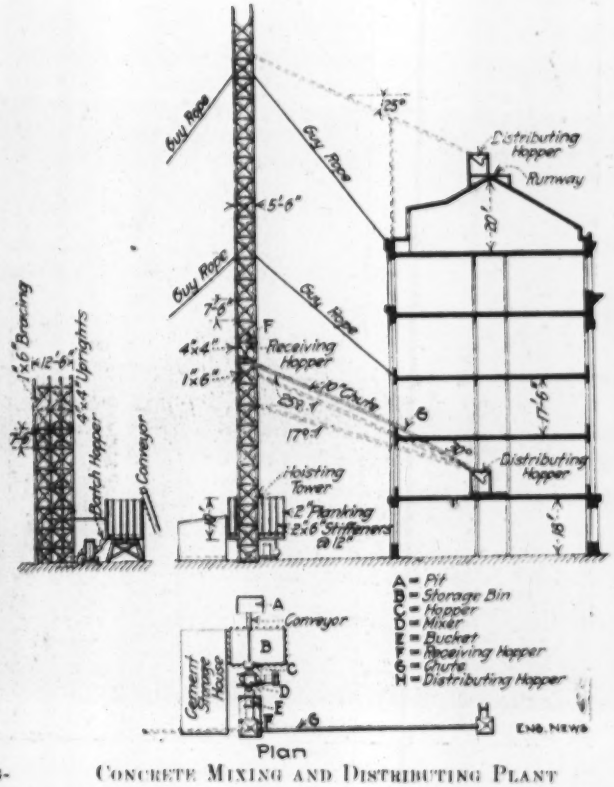


FIG. 2. PROGRESS DIAGRAM OF CONSTRUCTION

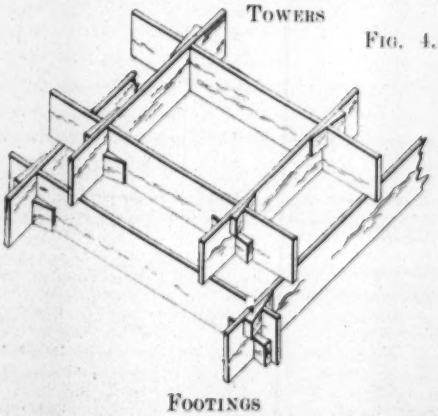


LAYOUT OF TRACKS AND DERRICKS AND CONCRETING-TOWERS

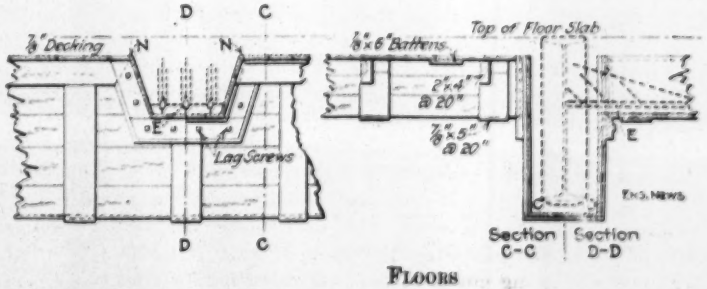


CONCRETE MIXING AND DISTRIBUTING PLANT

FIG. 4. PLANT LAYOUT FOR CONSTRUCTION



FOOTINGS



FLOORS

FIG. 5. TYPICAL CONCRETE FORMS

lowable. The design of the girder and beam forms is shown by Fig. 7. The forms being only tacked together with finishing nails at the points *N* they were readily removed without damage and used with little change for all floors.

During the warm weather in which the greater part of the concrete was placed, the forms were removed after a period of 14 days.

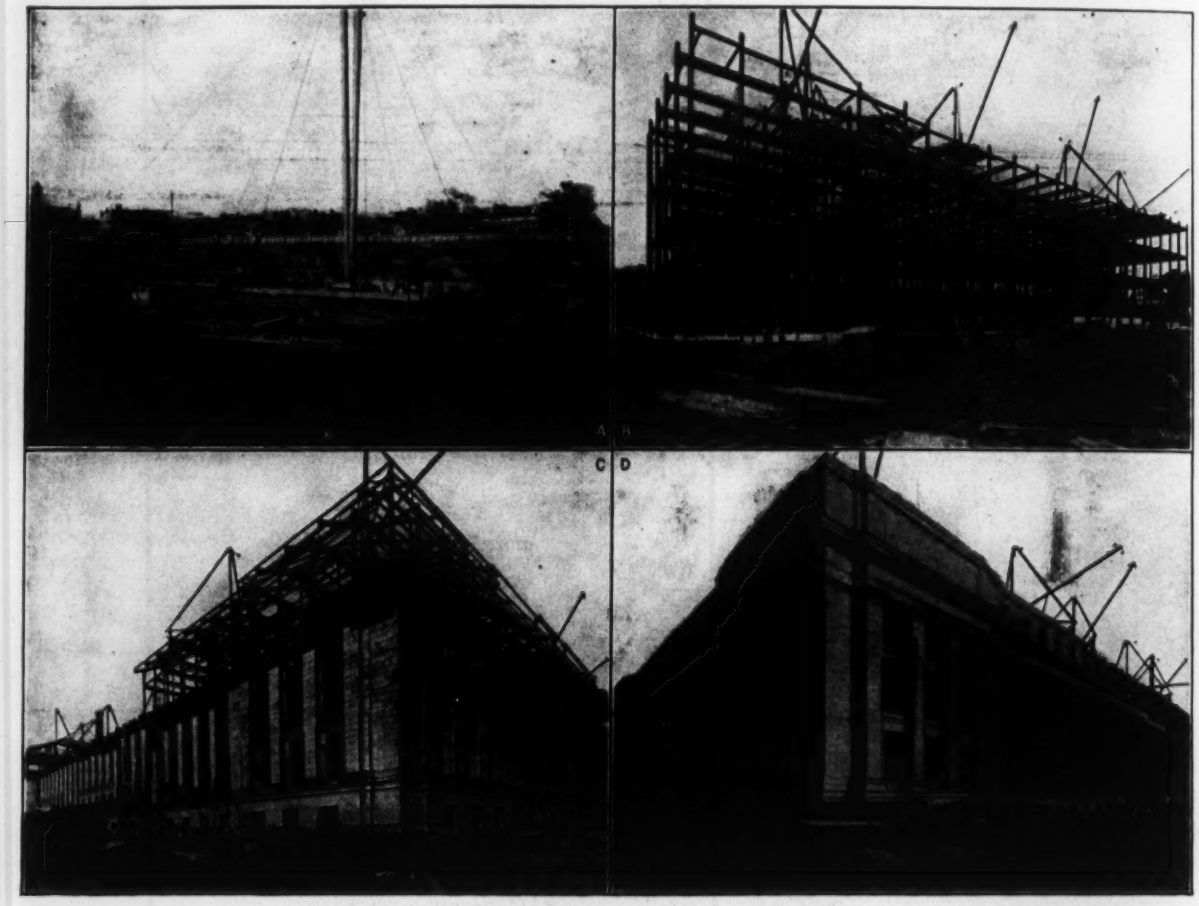
At the age of six months, a test load of 62,400 lb., or 1200 lb. per sq.ft., consisting of gravel in bags not bonded together, was placed on one of the Vault concrete beams, by which load a stress of 24,500 lb. per sq.in. in the steel and an extreme-fiber stress of 1080 lb. per sq.in. in the concrete were developed; the deflection of the beam was $\frac{1}{16}$ in., returning to normal upon the removal of the load. The 3-in. slabs of the floors in general were tested to 450 lb. per sq.ft., with $\frac{1}{32}$-in. deflection in the beams.

Steel window frames and sash are used in all of the courts and in the majority of other window openings. There are about 30,000 lights in these windows.

Some 600 carloads of Indiana limestone was cut at the quarry and set in about six months. A large quantity of the stone was placed on the walls directly from the cars, thereby saving much of the usual handling and risk of breakage. The twenty-five 18-ton lintels of the main colonnade were handled by a special 20-ton guy derrick, which placed four lintels at each set-up.

The compressed-air plant installed for the riveting was further effectively utilized in the cleaning of the limestone in place by means of air-drill motors and soft wire brushes.

J. Henry Miller, Inc., of Baltimore, Md., has the general contract for the building. The Furst-Kerber Cut Stone Co., of Chicago; the General Fireproofing Co.; the American Bridge Co.; and the Trussed Concrete Steel Co. were the subcontractors for the limestone, concrete, structural work and the steel windows, respectively. The writer was Superintendent of Construction for the office of the Supervising Architect, Treasury Department.



FIGS. 6-9. PROGRESS PHOTOGRAPHS OF NEW BUILDING FOR BUREAU OF ENGRAVING AND PRINTING, WASHINGTON, D. C.

(In first picture, the old building of the Bureau, where all stamps and paper money are printed, is at the extreme left.)

The Cost of Operation of 5-ton Motor Trucks is summarized as follows by the Locomobile Co., of Bridgeport, Conn. The fixed charges against the truck are estimated as follows: Garage, $25 a month; driver, $20 a week; insurance, $190 a year; interest, at 5%, $125; making the total fixed charges per annum, $1655. Taking 300 days to the year, this brings the fixed charges to $5.50 per day. The variable expenses are, depreciation, tires, repairs, gasoline and oil. By taking the average performance of many trucks the expense for these items per mile has been determined at 20c. Therefore in order to find the cost of operating a locomobile 5-ton truck, simply take $5.50 as the fixed charge and add 20c. for every mile the truck has traveled during the day.

A Large Reinforced-Concrete Culvert, Newcastle, Eng.

A very large and long reinforced-concrete culvert to carry the waters of a creek under a high fill is now under construction in the City of Newcastle, Eng. The work was described by Frank I. Morton, in a paper read before the meeting of the Institution of Municipal and County Engineers, at Newcastle-on-Tyne. We give below an abstract of that portion of the paper that relates to the construction and design of the culvert.

Two sections of the City of Newcastle are separated by a deep and wide valley called the Ouseburn Valley, at the bottom of which the Ouseburn Creek or river flows to the River Tyne. The only direct means of communication between the two parts of the city had been until recently, a brick viaduct, which, along about 1900, became too small for the traffic it was required to carry. At that time studies were made into a method of crossing the valley, and it was decided that it would be better to fill the valley for a length of about half a mile in the more congested part of the city, and to carry the necessary streets across the fill, using the remainder of the made land for ordinary building purposes. As the valley is about 100 ft. deep, with a maximum width of 1200 ft. and a minimum of 550 ft., the filling operation was to be a very extensive one, it being estimated that about four and one-half million yards of earth would be required.

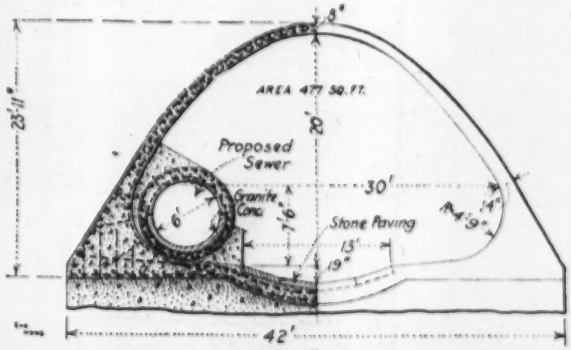
In order to spread the project over a long time, and thus reduce the cost, it was decided to utilize for the fill only the ordinary dump from excavations procured in the various building operations around the city and to start the work at one street, which was to be brought up to the required level before any additional filling was attempted. In 1906 this filling was started. It is an interesting commentary upon English methods to note the time which it is estimated will be required to complete the work. It is stated that the authorities hoped to be able to fill the embankment to obtain a road 20 ft. in width in about five years and a road of full width of 60 ft. in about eleven years, the entire filling to take a total period of 26 years.

It was necessary in making the fill to provide some method of caring for the waters of the Ouseburn, which drain an area of about 25 sq.mi. This creek has a very low summer flow but is subject to sudden floods. There did not seem to be any very good method of computing the required area of the culvert on account of lack of proper records, but it was decided, when the work was started in 1906, that a rainfall of 1 in. per hr. over the whole drainage area should be provided for. This would require an area of culvert of 477 sq.ft., assuming a 1 to 141 grade and a discharge of 16,080 cu.ft. per sec. The proposed fill was some 2000 ft. long and the culvert was to have that length.

To carry this flow the culvert shown in the accompanying cut was designed by the Hennebique Company and construction according to this design was begun in 1906. The culvert was founded on a solid concrete footing deposited in the excavated and unwatered stream bed. Special attention was paid to the reinforcement, which is shown in position but not in amount on the drawing, in order to make the culvert safe against the varying conditions of systematic filling over a term of years. It was not designed, however, to take flow under pressure.

Extensive experiments were entered into by the engineer with small models of the culvert to discover, if possible, the amount of the load of the earth fill transmitted to the arch. He proved to his own satisfaction that it would be necessary to design the culvert only to support 40 ft. of filling above the crown of the arch, the total filling being about 100 ft. Notwithstanding the results, however, the Local Government Board required provision to be made in the design for a total height of 80 ft. of filling becoming effective upon the culvert as a direct load.

After three years' work on the filling and on a small portion of the culvert immediately under the street being filled, it was found that the cost of land damages was excessive and that the amount of excavated material available in the City of Newcastle was too small to complete the work and in the time figured; so it was decided to review the work, and to provide, if possible, a new and less expensive design. The then city engineer reported



A Reinforced-Concrete Culvert to Carry 16,000 Sec.-Ft. under a 100-Ft. Fill

that the culvert as designed was probably much too large for the amount of flood water which it would ever have to carry. He recommended, therefore, that a culvert be designed with an area of 304 sq.ft. and that the filling be confined to two streets until such time as they were brought to a proper grade, when the filling could be carried into the proposed addition. The remainder of the culvert is now being built to this new size, the total length of the two being about 2060 ft., with a plan made up of a number of curves and tangents.

The smaller culvert is practically the same as the larger in design and shape. As will be noted, there has been provided in the design a proposed sewer 6 ft. in diameter at one side of the culvert. This sewer has not as yet been built, but rods have been left projecting from the side of the culvert to which the reinforcement of the sewer can be placed when it is required. The invert in the middle of the bottom which is paved with stone is to carry the low water flow only. The remainder of the invert is not paved. As stated before, the total area of the large-sized culvert will carry about 16,000 cu.ft. per sec. The smaller culvert will carry about 8700 cu.ft. per sec. including the sewer and about 6800 cu.ft. per sec. without the sewer. These figures, according to the latest engineer's report, are amply sufficient to take care of all the flow in the creek.

The Turbines of the Mississippi River Power Co. at Keokuk, Ia.

The great water-power plant at Keokuk, Ia., utilizing the power of the Des Moines Rapids of the Mississippi River, has just begun the delivery of electric current to St. Louis. For its successful completion on record time, notwithstanding its enormous size, great credit is due to the projector and Chief Engineer of the enterprise, Hugh L. Cooper.

A general description of the great work in its early stages of construction was given in our issue of Sept. 28, 1911. Among the many interesting features of the work are the hydraulic turbines which rank among the largest ever built. Each of these turbines is designed to develop 10,000 hp. with a head of 32 ft. The combination of large power and low head compelled very large machines. Thirty of these units are eventually to be installed.

FIG. 1. GENERAL VIEW OF THE TURBINE RUNNER

The weight of the Keokuk turbines, including their casings, approximates 500 tons each, while the aggregate weight of the rotating parts of each turbine, together with the generator attached thereto, is about 275 tons. The speed will be constant at 57.7 r.p.m.

GENERAL REQUIREMENTS

The turbines used in this installation are of the reaction type, and in order to understand why this type was chosen, we will consider the conditions affecting the choice and how the reaction type meets these conditions.

The most efficient electrical operation requires that the speed be constant and of as high a value as practicable. The reaction turbine is designed to operate with the buckets filled, and can be set low enough to utilize the entire head at all times and will operate efficiently when fully submerged. As the speed varies with the pressure and consequently with the head, it is evident that this type will have the highest speed. It also has a better speed regulation.

Before the design of these turbines was decided upon, competitive designs were secured from Norway, Germany, Switzerland, France, Italy and the United States. Those of the last nation seemed most nearly to meet the particular requirements. A committee was therefore formed consisting of the chief engineers of the two American manufacturers and the chief engineer and the mechanical engineer of the power company, to develop a new design, based upon the best ideas from all of the designs submitted. It was the function of this committee to decide not only upon the design of the turbine itself, but to select the best method of getting the water to and away from the turbine.

Before discussing the Keokuk turbines specifically, let us consider the essential differences between the American and the European types of reaction turbines. Broadly speaking, they differ in the shape and the dimensions of the buckets. The aim of this type of turbine is to secure in the runner, or rotating member, the largest possible discharge capacity, with the smallest diameter. This is desired because it will mean the minimum quantity of metal to utilize the largest percentage of the available reactionary force, and to obtain the highest rotational speed. Although decreasing the diameter increases the velocity, the head being constant, it decreases the discharge capacity, thereby defeating that important factor.

A large discharge capacity is secured in the American-type runner by the shape of the buckets and the axial depth of the entry passages; the latter are made as large as possible, not by increasing the diameter but by making the vertical sections of the vanes deeper. The outflow sections are enlarged by the broad dishing of these vanes. The latter scheme is illustrated by Fig. 1. In this manner the runner is given, in proportion to its diameter, abnormally large entry and exit areas, while the curvature of the vanes is such that the greatest amount of reaction force is utilized. The problem of securing the best entry and exit for the water is one requiring careful attention if it is to be solved efficiently. A poor solution has been known to reduce the efficiency of a plant by as much as 20%.

THE KEOKUK TURBINES

Fig. 2 is a cross-section through the main unit and illustrates how the water is received into and discharged from the turbine. The purpose of this drawing is to give a comprehensive idea of the design and operation of these turbines, and to that end most of the dimensional figures are omitted. There are 20 buckets cast radially around the central hub. The hub and buckets comprise the runner, which is 15 ft. 7⅞ in. in diameter. In an article elsewhere in this issue, by Mr. H. J. McDermid, it was shown that the two manufacturers who took the contract to build the turbines were compelled by the respective conditions of transportation to use different methods of casting the runners. The Wellman-Seaver-Morgan Co., of Akron, Ohio, who supplied seven of the fifteen units to be installed at the present time, cast each runner in one piece, weighing 67 tons. I. P. Morris & Co., the other manufacturer, who furnished eight units, cast each of its runners in four pieces, weighing, assembled, 95 tons.

The main shaft to which both the runner and the rotating element of the electric generator are direct-connected, is 25 in. in diameter, and has an overall length of about 24 ft. Thus the energy of the power developed by the buckets is transmitted directly to the generator.

The great weights mentioned suggest that the problem

lubrication was one of first importance. In spite of the length of the shaft it has been supported on one thrust bearing only. The difficulties of distributing the weight over several points of support and of maintain-

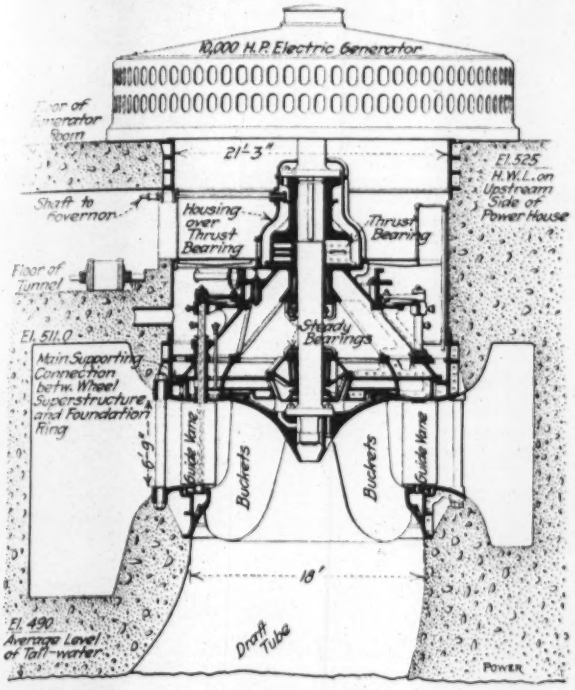


FIG. 2. CROSS-SECTION OF THE TURBINE

ing their alignment, have thus been avoided. The entire weight of the revolving parts is concentrated on the thrust bearing, as shown in Fig. 2. Several accidents in connection with vertical turbines have not continued this type in the best graces of engineers. The thrust bearing usually carries the weight solely by means of a water cushion, but the pressure which maintains this water cushion has been known to fail in certain instances.

In the case of the Keokuk turbines, the vertical arrangement is the only one practicable. Hence, it became necessary to evolve some scheme for minimizing the possi-

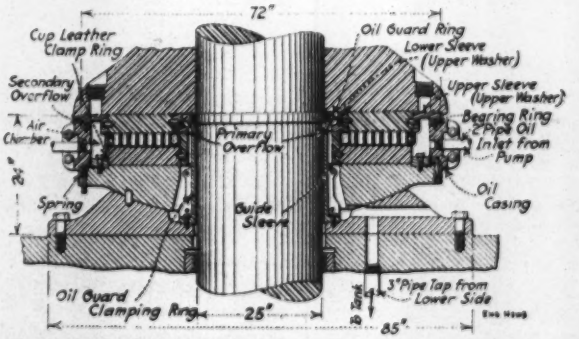


FIG. 3. METHOD OF SUPPORTING THE SHAFT

bilities of such an accident. This was accomplished (Fig. 3) by devising two independent methods of supporting the weight. The first was by forcing oil under a pressure of 250 lb. per sq.in. between two circular bearing disks, one stationary and the other rotating with the shaft. Thus the rotating part turned on a film of oil. The auxiliary method consisted in providing the thrust bearing on the inside with a set of steel rollers, which require merely to be immersed in oil in order to serve their purpose. As the housing over the thrust bearing is oil tight, even in the event of an interruption to the oil supply, the oil in the bearing at that time could not escape. Should the disks fail to receive the oil, the load will settle down until it rests on the rollers and will travel upon them until the oil pressure can be restored. The thrust bearing rests upon a conical casting which in turn is supported on a foundation ring, indicated at about the center of Fig. 2.

All of the auxiliary parts incident to the turbine lubrication are located within a cylinder, 21 ft. 3 in. in diameter, built up as shown in Fig. 2. The guide vanes are maintained an exact distance apart at all times by means of twenty 8-in. bolts, connecting the two castings, *H* and *I*. Thus these vanes cannot pinch and work hard in operation.

Previously, turbines of this design and size have had a part of their bearings running in water, so that lubrication could not be applied except through the medium of the water, thus largely increasing the cost of maintenance. The use of one runner on the main shaft, instead of two runners, as previously used for this amount of power and head, reduces the cost of operation and maintenance by about one-half. This largely decided the selection of the single-runner type.

THE METHOD OF SPEED CONTROL—When a certain percentage of the load which the turbine is carrying is thrown off, it is necessary that there be some method provided for instantaneously and automatically throttling an amount of water which the buckets are receiving, equivalent to the given percentage of the load removed. This throttling is accomplished by changing the position of the guide vanes, shown in section in Fig. 2, and in plan in Fig. 4. There are twenty guide vanes, each operated by a vertical independent shaft. Each of these shafts is connected by cranks to a rocker ring, which revolves horizontally through a short arc, sufficient to open and close and give intermediate openings to the guide vanes. In this way the amount of water flowing between these guide vanes onto the buckets, is regulated. Any tendency of the generator to change its speed is detected by a flyball governor. As an emergency precau-

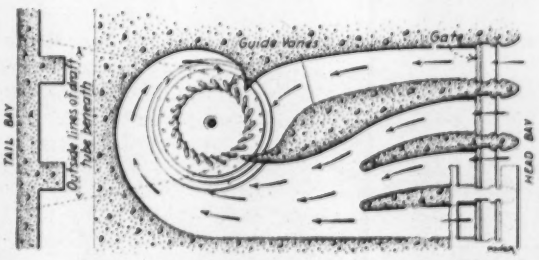


FIG. 4. METHOD OF CONDUCTING THE FEED WATER TO THE RUNNER

tion, should the whole load be suddenly thrown off the generator, the governor is so designed that it will automatically close the guide vanes. Fig. 4 shows the arrangement of the guide vanes around the periphery of the

runner. In this figure the vanes are open to their normal position. The dotted lines show the position of the vanes when closed.

THE INTAKE AND DISCHARGE PASSAGES—The problem of moving the water from its highest elevation at pool level above the dam, through the turbines and out into the main river again, to an elevation of 35 ft. less, will now be considered.

The rate of supply of the water to the runner should be uniform all the way around its periphery. This requires that the shape of the water conduit, which conveys the water around the periphery, must constantly decrease in cross-sections. This condition is illustrated in Fig. 2. The section B, B, B, B at the left of the figure is large as compared to the area B_1, B_1, B_1, B_1 to the right. When the water conduit reaches the last guide vane it is reduced to its smallest section.

The runner demands this uniform supply in order to maintain its highest efficiency, for proper lubrication, and for the maintenance in line of the shaft and the steady bearings.

Having designed the manner of intake, the next requisite is a proper design for the outlet which is to carry the water away from the wheel and back into the river. This process should involve the least loss of energy. The draft tube or discharge passage at a point where the water first enters has a diameter of 18 ft., and under normal conditions the velocity of the water past this part will be 14 ft. per sec., and upon passing out of this same tube into the river, the conduit is so enlarged that the discharge will be at the rate of about 4 ft. per sec. This reduction of velocity through a tube of such shape as the one here described saves to the total normal head of 32 ft., 2½ ft. of effective pressure.

Ten of these turbines have been delivered at the power plant at the present time, and five more are under construction. The completed station will contain 30 of these units and four 2500-hp. exciters of the same general description as the main units.

Water was admitted to one of the main turbines, and power was furnished, for the first time, on June 4, 1913.

For the data from which the above description is prepared we are indebted to some of the excellent illustrated Bulletins which have been issued at monthly intervals by the Power company during the progress of the work.

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The Turbine Runners of the Mississippi River Power Co. at Keokuk, Iowa

BY H. B. MCDERMID

At the Keokuk hydro-electric power plant of the Mississippi River Power Co., now nearing completion, each turbine is designed to develop 10,000 hp. at a 32-ft. head and will weigh, complete, about 500 tons. The runner, or rotating member carrying the blades, weighs either 67 or 95 tons, depending upon its manner of construction. The huge dimensions of these runners raised difficulties not only in design, but also in transportation. It is the purpose of this article to describe how the latter difficulties were overcome.

Owing to the heavy requirements of the turbine contract it was divided between two companies. One of these, the Wellman-Seaver-Morgan Co., of Akron, [illegible], who contracted to supply seven of the fifteen units to be installed at the present time, was able, by virtue

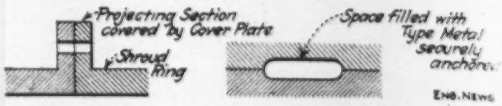


FIG. 1 FIG. 2

of a favorable location with respect to Keokuk, to cast each runner in one piece. The other manufacturer, the I. P. Morris Co., of Philadelphia, having its transportation problem complicated by several tunnels, was forced to adopt the plan of casting the turbine runner in four parts, a most difficult proceeding due to the double curved surfaces of a Francis turbine vane. To make this method a success it was necessary that the joints should be made with sufficient rigidity and should present smooth surfaces to the action of the water. If this were not done, hydraulic disturbances would preclude the attainment of the highest efficiencies.

The job was accomplished by cutting the upper shroud ring in a vertical plane along the lines of the upper ends

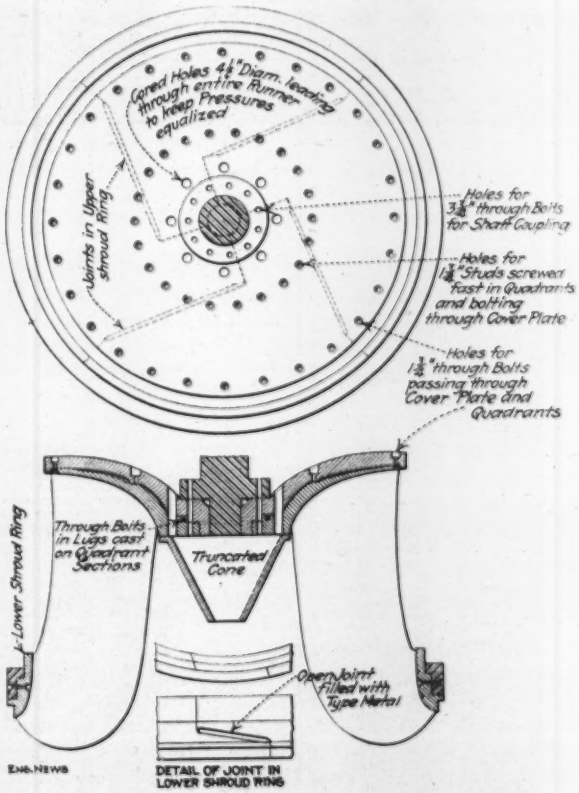

Cored Holes 4½" Diam. leading through entire Runner to keep Pressures equalized
Joints in Upper Shroud Ring
Holes for 3½" through Bolts for Shaft Coupling
Holes for 1¾" Studs screwed fast in Quadrants and bolting through Cover Plate
Holes for 1¾" through Bolts passing through Cover Plate and Quadrants
Through Bolts in Lugs cast on Quadrant Sections
Truncated Cone
Lower Shroud Ring
Open Joint Filled with Type Metal
ENG. NEWS
DETAIL OF JOINT IN LOWER SHROUD RING

FIG. 3. ASSEMBLY DRAWING OF THE TURBINE RUNNER
(Built by I. P. Morris & Co., Philadelphia, Penn.)

of the runner vanes, and introducing in each joint two through bolts passing through lugs cast on each piece (Fig. 1) for partially holding the parts in place. These lugs are afterward covered by a cover plate so as to avoid all hydraulic disturbances. The lower shroud ring was cut in three planes, one following the vane lines as nearly as a plane surface can be said to follow a curved surface,

the other two being vertical planes—one at each end of the inclined plane first mentioned. Angle straps were then attached to each piece to aid in drawing up the joints temporarily until an outer ring or hoop could be put in place.

In each case the joint line is cut away as per sketch (Fig. 2) to allow filling with type metal which is solidly anchored in place by grooves cut in the center of the section of the joint. In the case of the lower ring, where the type metal is introduced in the cut made on the inclined plane lying between the two vertical planes of the joint, where the surface cannot readily be machined accurately, this method is much cheaper and makes fully as substantial a joint as one made iron to iron. In the case of

FIG. 4. GENERAL VIEW OF THE RUNNERS

the upper joint, which is all in three vertical planes, the type metal, by means of its large anchor grooves, will effectually prevent any slight vertical play of the joint which might otherwise occur.

Both ends of the runner parts were then rigidly secured in place by means of solid rings completely encircling them. The enveloping ring for the lower shroud ring is of steel, machined to a snug fit and further secured by horizontal radial bolts which pass through both rings. Their special round heads are countersunk so that they present a smooth flush surface on the inner side of the shroud ring. The nuts are contained in countersunk recesses in the outer surface of the enveloping ring. After the nuts are tightened, these recesses are filled with type metal in order to keep a smooth surface on the outside of the wheel.

The upper ring is secured in a similar manner except that the bolts are passed vertically through the shroud ring and a cover plate, which has a male and female fit upon the quadrants containing the vanes. This plate serves to cover the joints shown in Fig. 1 and also to act as a support for the shaft and, with its external fit, to hold the quadrants rigidly together. Examination of Fig. 3 and the photographs (Figs. 4 and 5) should make the design apparent. The whole effect is to make the wheel to all practical purposes, as smooth in its hydraulic surfaces and as rigid as though cast in one piece. This has been accomplished and the runner operates satisfactorily in spite of the fact that it is composed of seven principal parts—the four quadrants, each carrying four vanes, one cover plate, one ring enveloping the lower shroud ring, and one truncated cone which is bolted inside the vanes underneath the end of the shaft to further guide the discharge water in the same direction it has followed along the lower surface of the upper shroud ring as per Fig. 3.

The weight of the built-up wheel is considerably greater than that of the one cast integral, weighing approximately 95 tons in comparison to the 67 tons of the latter. Both are nearly identical in size, the greatest difference in design being merely that due to the different methods employed by the two manufacturers in transporting their wheels to the power plant.

Let us now consider the other manufacturer who had merely to cast his runner in one piece. The low head and high power requirements necessitated very large dimensions for the runners. Each is a trifle over 16 ft. in diameter at its lower end and approximately 12 ft. in its greatest vertical dimensions. It is, therefore, evident that the manufacturer who chose the method of casting his wheel whole had no small problem before him from the standpoint of transportation. It was solved by designing and building a special steel car, so made that the runner could rest upon its side in a steel cradle, machined to fit its upper and lower shroud rings. The wheel was rigidly secured to this cradle by steel bands, passing over the wheel and supplied at each end with a bolt and nut which could be tightened as desired. A match mark accurately made on wheel and cradle before starting, showed at the end of the journey that the wheel had not stirred upon its bed during transit. It was necessary to proceed with great care with such a load, extending as it did some distance on each side of the rails, and while clearing the latter but a few inches, towering above them over 16 ft. The first trip was made entirely by daylight under the constant supervision of the manufacturer's traffic

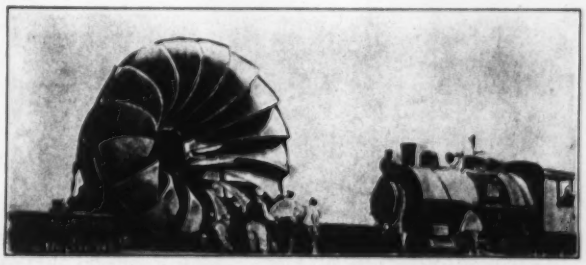


FIG. 5. VIEW OF THE TURBINE RUNNER AND THE SPECIAL CAR
(Built by the Wellman-Seaver-Morgan Co., Akron, Ohio.)

manager, while in many instances special provisions for obtaining the necessary clearances had to be made. The car, with the runner in position upon it, is shown in Fig. 5.

As a proof of the sound judgment governing the selections of the two modes of casting and construction, it should be stated that at the present time seven of each of these types of runners have been delivered without injury and nearly all of these have been installed in their permanent position.

Typhoid-Fever Deaths in New York City for the second quarter of 1913 totaled only 39. Allowing for differences in population, the average for the same quarter during the preceding five years was 91.

The Georgia-Harris Viaduct, Vancouver, B. C.

Construction has recently been commenced on a very long viaduct of somewhat peculiar design on the line of Georgia and Harris Sts. in the City of Vancouver. The crossing has been under consideration by the city for a number of years to connect the east end of the city more directly with the business district. The location is across a narrow arm of English Bay, known as False Creek, and a valley which contains the tracks of the Great Northern, Canadian Pacific and British Columbia Electric railways. About four years ago it was estimated that the construction of a viaduct at this point would cost about $550,000 and a bylaw providing this amount was voted upon favorably by the citizens of Vancouver in the early part of 1912.

Immediately thereafter a call for competitive designs from different engineers was issued, on a viaduct to be approximately 2860 ft. long over the valley noted, with a total width of 66 ft., made up of a 53-ft. roadway and two 6½-ft. sidewalks, the whole structure to be either of steel or concrete and to be designed to carry heavy interurban railway cars. Prizes for the competition were offered as follows:

First Prize—Acceptance of plans with a 5% commission for supervision and design.

Second Prize—$750.

Third Prize—$500.

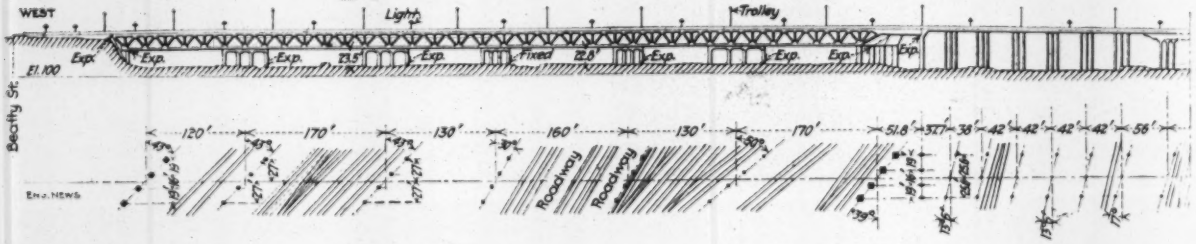
Fourth Prize—$250.

On June 28, 1912, twelve designs were received from nine Canadian and American engineers with costs ranging from $402,000 to $611,000. After a careful investigation of the designs submitted the city awarded the first prize to C. A. P. Turner, of Minneapolis and Vancouver, the second prize to Waddell & Harrington, of Kansas City, Missouri, the third prize to E. A. Stone and the fourth prize to Wilson, Lake & Benthall; the two latter are both Vancouver firms.

Mr. Turner submitted two designs, one using his "mushroom" system of flat slabs as adapted to bridge spans for the entire crossing, with the exception of an arch span over the creek; the other design, which was accepted and is shown in the accompanying line cut, was made up of a great variety of different types of structure. The former design was estimated to cost $402,000, while the latter was estimated at $463,600. These estimates were guaranteed by the designer who also furnished the usual guarantee for the strength of the "mushroom" construction. Tenders for the work were received on Jan. 30, 1913; the two lowest bidders were well within the engineers' guaranteed estimate and varied from each other less than ½%. The lowest figure was received from Martin Carroll Co., of Kansas City, Missouri, and the next lowest was James McDiarmid & Co., of Winnepeg and Vancouver. The contract was awarded to the latter, the city showing a preference to the local and Canadian firm.

An outline elevation of the structure and a plan showing the location of the piers and columns is given in the accompanying cut. Beginning at the west side there is a short approach, and then about 880 ft. of structure carried on four continuous Warren trusses with the steel members incased in concrete to conform with the rest of the structure and to resist the action of the locomotive gases from the trains passing underneath. The trusses are continuous over the entire length, but rest on concrete-incased steel bents so spaced as to form spans varying from 120 to 170 ft. and on different angles of skew to conform with the location of the railway tracks.

These column bents are of steel, framing directly and rigidly into the upper and lower chords of the trusses, of which there are four in transverse line. Expansion is provided for at the base of all column bents, except the central one as indicated on the cut, by roller bearings resting on plates after the usual design. The concrete floor of this portion of the structure consists of a 2-way system resting on the tops of the trusses and transverse beams provided with an overhang for the sidewalks.



THE GEORGIA-HARRIS REINFORCED-

(A succession of concrete-encased steel Warren trusses, "mushroom" slabs, reinforced-

Each end of this continuous truss is in cantilever form extending on the west side to the approach and on the east side to a short reinforced-concrete girder span which transforms the skew into a square location of the piers. This span is composed of four reinforced-concrete girders each provided with a peculiar reinforced-concrete strut member framing into the overhang of the truss. The east end of the girders rests on a bracket in a reinforced-concrete column and provides there a sliding expansion joint.

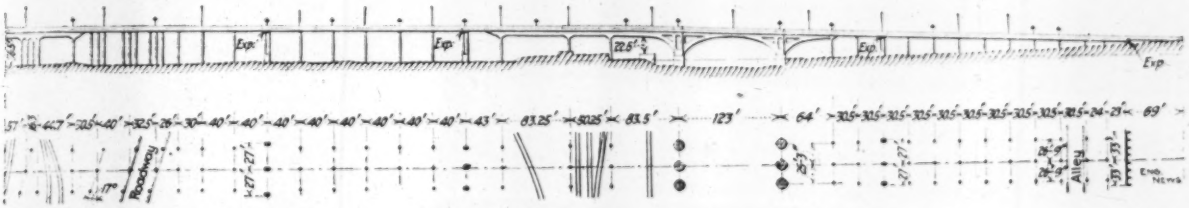
There follows then about 300 ft. of flat-slab construction after Mr. Turner's well known "mushroom" design. The 300 ft. is made up of about seven spans varying from 38 to 43 ft. 6 in. in length each supported on three reinforced-concrete columns on a skew to fit the tracks below, and resting on solid footings and carrying at the top the usual mushroom head and flat slab peculiar to the design. These columns are in some locations as long as 50 ft.

At the end of this stretch of "mushroom" slabs there are two girder spans made up of three reinforced-concrete girders carrying cross-beams and floor-slabs, and extending at either end in cantilever to meet the floor slabs of

the adjacent "mushroom" slabs. At this location the requirements of the railroad tracks demanded a span greater than the present longest "mushroom" span, which is 50 ft.

After the girder spans there are 14 more "mushroom" flat spans, three of which are on the skew and 11 on the square, and each carried on bents of three reinforced-concrete columns. In the middle of this stretch of spans there is a larger pier provided on which the slab slides to form an expansion joint, which is the only expansion joint in this part of the structure for 600 ft. This is somewhat longer than the usual practice but it is thought that the variation of temperature in Vancouver is so little that the extreme length will not be too great.

The extreme end of this "mushroom" slab system frames into another set of reinforced-concrete girders, the largest of which has a span of 83.25 ft. These girders are succeeded by what appears to be a succession of arch and half-arch spans but what is in reality two cantilever spans resting on cylinder caisson piers and framed to represent arch spans of 83½, 123 and 64 ft. respectively. These spans are reinforced with structural-steel reinforcement with the framing sufficient strong to carry the dead-load during construction, and with additional reinforcing rods to carry the live-loads.



CONCRETE VIADUCT, VANCOUVER, B. C.
concrete girders and steel-frame cantilevers. Designed by C. A. P. Turner.)

The eastern end of the bridge is made up of 12 mushroom spans with one expansion joint and a straight wall at the eastern abutment. The embankment at western end is about 40 ft. high and has stood a number of years in an almost vertical position. The formation of the bank is a hard blue clay which only requires protection from the weather in order to retain its position, so an inclined wall strengthened with the large piers for the westerly supports for the continuous trusses has been designed to form this abutment.

Starting from the west the bridge has a 1.01% grade up to the end of the truss system, and a 1.15% grade from there down to the beginning of the cantilever arch over the creek whence there is a 3.29% grade to the east.

The ornamental features of the viaduct consist of a high-grade artificial cast-stone lamp standards and hand rail. A block pavement will be laid on the concrete slabs to form the roadway.

F. L. Fellowes is Supervising City Engineer of the City of Vancouver, and is in direct charge of the construction of the bridge. The local manager for the contractor, James McDiarmid & Co., is T. Bortford and A. P. Hueckel is resident engineer for C. A. P. Turner, the chief consulting engineer on the work.

We are indebted to Mr. Hueckel for the information and drawing which are presented herewith.

Economy in Road Location on Sandy Soils

BY ROLAND WATTS*

In constructing roads in flat, sandy parts of the country, some of the State Highway departments attempt to secure grades of at least 0.5% for proper drainage. This rule often makes very heavy fills necessary. In some cases stretches from four to five miles long are practically all fill; the idea being "to lift the road up out of the water."

The material for these heavy fills must be obtained from borrowpits, often at heavy expense. I recall an instance, in one of the Atlantic Coast States, where about 2500 cu.yd. of dirt had to be hauled two miles to make a fill; and the cost of the borrowpits and overhaul alone on this job was about $2000.

In a great many cases "lifting the road out of the water" is not necessary. I have seen some heavy fills made over some material depressions and culverts put in at the low points, but no water ever runs through the culverts as the soil is sandy, and absorbs the water like a sponge. I have gone over some of these roads after exceptionally heavy rains; and have noticed a number of long, low places over which fills had been made, some of them nearly a mile long, to which apparently the water drained from all directions from quite an area, and in most cases the water has seeped away within one or two hours.

This gives rise to the thought that much unnecessary expense could be saved by the engineer in making the preliminary inspection of the road. Instead of following fixed rules regarding minimum grades, etc., spend more time in noting the soil conditions. Before recommending a heavy fill over a long, low place, dig test holes. If much sand is struck, we may be able to secure all necessary drainage by digging side ditches.

If the soil is very sandy it would be safe to run practically flat grades; the water will never stand in the ditches long enough to damage the road. If the soil is not so sandy, make grades of ditches from 0.2% to 0.5% so that water which does not seep away will gradually drain off.

Water which stands for any length of time on any property drowns out the crops. Hence, if a low place on the line of a proposed road has healthy looking, thriving crops, it is pretty safe to assume that the soil is fairly porous and that water will never stand long enough to damage the road.

*Highway Engineer of Florence County, Florence, Wis.

Auxiliary Deep-Well Water-Supply with Electric Pumps

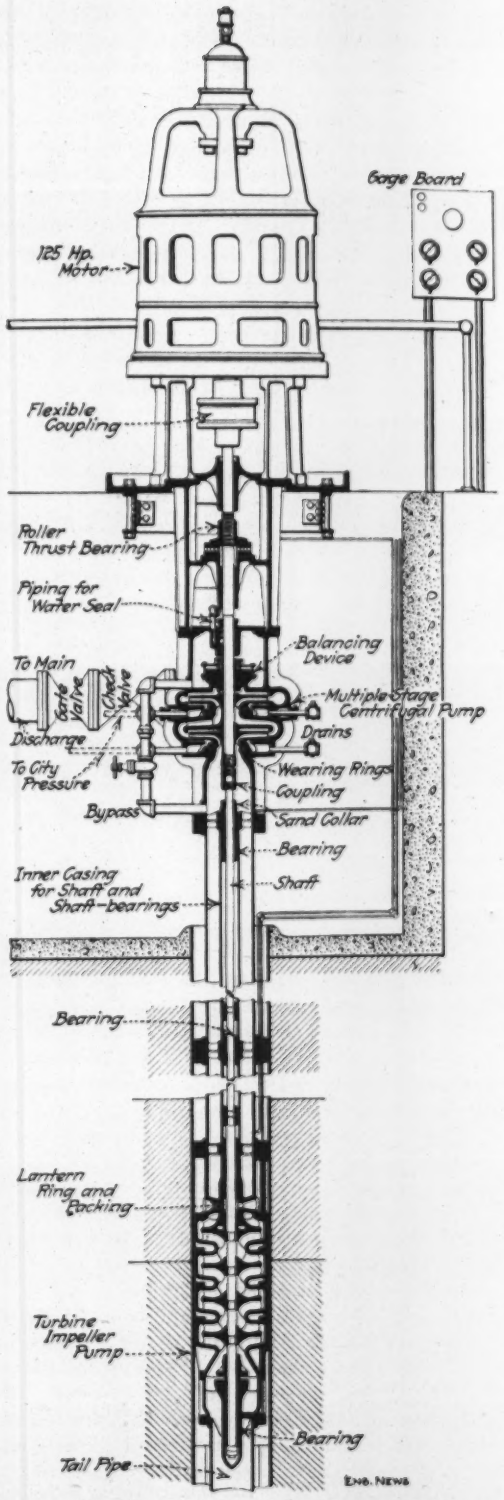
The main water-supply of the city of Rockford, Ill., is taken from deep wells in the Potsdam sandstone by means of collecting tunnels centering at a shaft nearly 100 ft. deep. At the bottom of the shaft are located centrifugal pumps with rope drive from vertical engines at the top of the shaft. These pump the water to reservoirs, from which it is pumped to the mains. This plant (described in ENGINEERING NEWS, July 13, 1899) was put in operation in 1898, the city having at that time a population of 30,000. The capacity (by test) was about 6,800,000 gal. per 24 hr., which was ample for both domestic service and fire protection.

The average daily domestic consumption ranged from 2,586,000 gal. in 1898 to 3,918,000 in 1904, and 3,565,000 in 1909. By 1909, the population had increased about 50% over 1898 (or to nearly 45,000), and the water-supply had become insufficient at times of maximum pumping. In 1910, the mayor appointed a commission to consider plans for increasing the water-supply and improving the distribution system, both for general supply and for fire protection. This commission was composed of the following hydraulic engineers: John W. Alvord (Chicago), Dabney H. Maury (Chicago), and Daniel W. Mead (Madison, Wis.).

The plan recommended by the commission, in 1911, included two separate features: (1) to provide an increased supply at once by an auxiliary well with electric pumps delivering directly to the mains; (2) to construct a new pumping station with a shaft and tunnel system similar to the existing system. The first part of this plan has been carried out and the auxiliary well and pumping plant put in service. This may be supplemented by other similar wells and pumps at different points where needed, and it was suggested that for economy these may be located on ground occupied by the city fire stations. They would be operated as required by the demand. The first well, however, will meet the requirements for some time to come. Nothing has been done in regard to duplicating the present shaft and tunnel system, as this appears to be unnecessary.

In regard to the auxiliary supply from the well, the commission proposed that the water should be raised by a deep-well pump and delivered into the mains by a pump at the surface. Additional pressure for fire service could be furnished by a third pump. All of these were to be motor-driven centrifugal pumps, with the fire pump so arranged that it could be started by throwing a switch and without opening or closing a single valve. The arrangement as carried out is shown in the accompanying cut.

The new well is about 1500 ft. deep (striking the Potsdam sandstone at about 930 ft.) and has a bore of 12 in., which is increased to 18 in. for the first 250 ft. At about 110 ft. from the surface is a 17-in. four-stage impeller pump mounted on a vertical shaft which is driven by a 125-hp. 440-volt motor standing directly over the well. Near the upper end of the shaft is a two-stage centrifugal pump which receives the water rising in the well and delivers it through a horizontal connection to the main. The shaft is supported at intervals by guide-bearings which are placed within an inner sleeve or casing, separating them from the annular water chamber of the well. The current is purchased from the Rockford Electric Co. and is stepped down from 4000 to 400 volts at the pump house. The plant is stopped and started automatically by means of apparatus controlled by varia-



WELL AND ELECTRIC PUMPING PLANT FOR AUXILIARY WATER-SUPPLY AT ROCKFORD, ILL.

...s of pressure in the main. The water is all passed through an 8-in. Worthington meter.

The pump house is a brick building 19x14 ft., with a slate roof. Besides the motor, gages and control apparatus it has an oil pump and filter system for lubricating the shaft bearings; this is driven by a motor of ⅛ hp. Above the well is a 5-ton trolley hoist for handling the machinery. It is proposed to provide an electric control from the main pumping station, as the pressure in the mains is raised to 85 lb. for fire service, and with the automatic control alone this would shut down the auxiliary plant.

The plant has a guaranteed over-all operating efficiency of 59½%, but on tests an efficiency of 65.5% has been shown. On these tests the delivery was 1380 gal. per min. against a total head of 237.5 ft. with a discharge pressure of 61½ lb., equivalent to 142 ft. When pumping out the well to clear it of sand, the pump delivered at the rate of 2,500,000 gal. per 24 hr., with a discharge pressure of 25 lb. at the 5-in. hydrant outlet.

The pumping plant was built by the American Well Works, of Aurora, Ill., and the work was done under the direction of G. G. Crane, Superintendent of Water Works, Rockford, Ill.

The Performance of An Oil-Engine Ocean Vessel

The oil-engine cargo ship "Christian X" of the Hamburg-American Line was described in our issue of Oct. 3, 1912. We give below some extracts from a report on the performance of this vessel on its regular voyages, which information is of interest and importance in view of the rapidly increasing use of oil engines for marine work. The ship is 370 ft. long, 53-ft. beam, 30 ft. deep, with a loaded draft of 23 ft. 6 in. and a deadweight capacity of 7400 tons. The twin screws are driven by a pair of eight-cylinder four-cycle Diesel engines, aggregating 2500 i.hp. at 140 r.p.m. and there are two similar auxiliary engines of 200 hp. at 225 r.p.m. The deck machinery, winches, windlass and steering gear are electrically driven. The ship was launched in March, 1912. The following account of its sea service is abstracted from a report published in *The Engineer* (London), July 18, 1913.

After loading in Hamburg for Havana, she commenced her first voyage on July 23, 1912, and until the vessel ran into Havana on Aug. 9, the engines ran at full power without any stoppage. The weather was very good, except for a couple of days when a fresh westerly wind raised a very rough sea, so that the propellers now and then came partly out of the water, causing the governors to come into action.

The fuel used was Roumanian oil. Its effective heat value was 17,800 B.t.u. The total consumption of fuel in 24 hours' trial was 8.545 tons (metric) for the main engines and 0.84 ton for the auxiliary engine. Thus the consumption per i.hp.-hr. was: Main engines, including the oil used for the auxiliary machinery, 0.361 lb.; main engines, excluding the oil used for the auxiliary machinery, 0.328 lb.; the auxiliary engine, 0.357 lb. At Havana, the machinery was overhauled and found to be in perfect order, though the exhaust valves were changed and the oil valves were ground in.

The ship then proceeded to Vera Cruz. In August an easterly trade wind blows at about the same rate as the vessel's speed, and this portion of the voyage was hottest of the whole trip in the engine room, since the ventilators did not carry much air to the engineers' platform. The highest temperature was on Aug. 15. The temperature on deck in the shade was 89.6° F., and that in the engine room 107.6° F., or considerably less than the temperature in the engine room and the stokehold of a steamer under similar conditions.

From Vera Cruz the ship proceeded to Tampico, and took 100 tons of oil fuel, which was said to contain 1.72% of sulphur. The engines worked excellently with this oil, although the exhaust gases smelt very strongly of sulphur. On this account, in order to run no risk of damaging the machinery by the action of the sulphur, it was decided to continue the voyage on the Roumanian oil which was still left in the bottom tanks, until the other oil could be analyzed in order to make sure that the proportion of sulphur did not exceed 1.72%, which, of course, would be harmless.

The vessel left Coatzacoalcos Aug. 31 for New Orleans with her holds empty. There it took a full cargo and left on Sept. 15, arriving at New York Sept. 19.

After filling the bottom tanks with fuel oil the ship left New York on Sept. 20 for the return to Hamburg. The next day there was a very strong head wind, the sea was very rough and the ship pitched and plunged heavily. The Aspinall governors worked without interruption as the propellers were thrown right out of the water. On Sept. 23 and 24 the starboard engine was put "half speed" to enable the ship to steer better against the high sea. On those two days the speed was only 6.26 and 8.41 knots respectively. On the 30th a storm commenced again from the northeast, and the engines had to be stopped for eight minutes to clean out the oil filters which were not then provided with bypasses. The constant rolling of the vessel set the oil in the tanks in such violent movement that the sludge or sediment had got down into the piping and had stopped up the filter entirely.

On Oct. 2 the very high sea smashed the railing on the promenade deck and bent all the awning posts on the port side. The starboard engine was afterward put to half speed so as to enable the ship to hold on her course, and the speed dropped to 5.9 knots. The vessel reached Hamburg on Oct. 6. The mean speed for the home voyage was 9.58 knots, a good result if bad weather and the head wind the whole way are taken into account. A summary of the voyage is shown in the accompanying table.

PERFORMANCE OF THE OIL-ENGINE SHIP "CHRISTIAN X" ON A VOYAGE OF 11,894 MILES

	Voyage days	Voyage hr.	Distance naut. miles	Speed knots per hr.	I.hp.	Oil consumed Total tons	Oil consumed Per 24 hp. tons	Oil consumed Kilos. per I.hp. per hr.
At Hamburg						14.02		
To Havana	17	12	4627	11.01	2390	179.80	9.732	0.169
To Vera Cruz	2	19	810	12.11	.....	38.75	10	.....
To Tampico	0	17	210	12.54	.....	10.28	.....	.....
To Coatzacoalcos	1	3	311	11.38	.....	13.28	9.50	.....
To New Orleans	2	10	698	12.10	.....	33.68	9.80	.....
To New York	5	5	1613	12.92	.....	58.60	9.90	.....
To Hamburg	15	18	3625	9.58	2415	157.00	9.713	0.168
Total and average	45	12	11894	10.89	2440	505.41	9.75	0.169

On arrival, the engines were inspected and everything was found in order. Of the escape valves, which had been working since the departure from New Orleans, only two were attacked to such an extent that it was necessary to turn the valve seats. Notwithstanding the bad weather on the home run, the mean speed over the whole trip, outward and homeward, was 10.89 knots.

On the second voyage of the "Christian X," which was made to New York and Philadelphia, things went well, and, notwithstanding very severe weather in the Atlantic on the outward trip, the average speed was 10.56 knots, while on the home run the rate was 11.41 knots.

On the third voyage, from the departure, on Jan. 6, a westerly hurricane and wild sea had to be fought up to Jan. 15, and according to the engineer's log books, the Aspinall governors were working uninterruptedly. One of the lifeboats, the after wheelhouse and various fittings on deck were washed overboard, and it became necessary to slow down the engines in order to prevent everything being swept away by the heavy seas that constantly washed over the vessel.

In New York a new sort of oil fuel was taken on board which caused too early ignition with the engines going slow and the Aspinall governors at work, so that the valves hung open and some of them were spoiled. As there were only a few spare fuel valves, and as the captain did not think that he could hold the ship against the strong sea with a single engine in case of need, he preferred to turn and put into Queenstown, Ireland, rather than expose the vessel to further damage. On arrival in port the ignition was retarded by a simple operation, and the ship then went out on a trial trip which showed that all was in order. Spare valves were put on board and the vessel then continued her voyage toward Boston, where she arrived Feb. 15 without trouble, but again after experiencing very severe weather.

The Cost of Reclamation Service and Other Irrigation Projects in Colorado

By John E. Field*

Within the last few months there have appeared in several of the engineering magazines certain articles entitled, "The Cost of Irrigation Works per Acre Supplied with Water." (See Engineering News, May 15, 1913.) The articles were practically identical, and as the U. S. Reclamation Service is credited with having furnished the data, it is obvious that it is, to all intents and purposes, an official statement from that bureau of the Government.

The writer does not hesitate to criticize the arti... as his connection with the Reclamation Service from ...3 to 1907 as project engineer on the North Platte ...l Pathfinder projects has given him an insight int... methods of the Government and the difficulties a... which it must contend. Six years' connection with ... office he now again occupies, and as consulting and ... structing engineer on several important private e... prises between 1907 and 1913, justifies him, he beli... in expressing opinions on private enterprises and in ...ing comparisons.

A large proportion of the disadvantages in connec...n with this Government work are inherent in all Gove...ment work and are not the fault of, nor can they ... corrected by, the individuals who happen to be in charge.

TABLE I. COST ($) OF THE COLORADO PROJECTS LISTED IN THE ARTICLE PREPARED BY THE U. S. RECLAMATION SERVICE.

Projects	Original sale price	Cost subsequent improvements	Total cost to users	Maintenance and operation	Present value	Bonds	Improvements needed	Area	Price given by U.S.R.S. Water	Price given by U.S.R.S. Water and land
Amity	10.00	53.50	65.50	1.00	75.00	....	....	45,000	....	100
Beaver Land & Water Co.	....	....	75.00	....	75.00	....	....	7,000]	....	175
Catlin	10.00	....	10.00	0.50	75.00	....	....	18,660	100	....
Colorado Coöp. Co.	....	....	....	1.40*	16.33	....	8.70	4,600	60	....
East Palisade†	....	....	61.50	....	....	....	....	645	63	....
Fort Lyon	10.00	15.00	26.58	0.40	50.00	8.50	1.58	68,000	100	....
Grand Valley	4.17	10.00	14.17	0.75	30.00	1.50	0	40,000	60	....
Greeley Poudre	40.00	....	40.00	....	....	....	....	125,000	45	....
Mesa County†	42.00	30.00	72.00	....	....	....	....	2,600	73	....
Orchard Mesa I. D.†	90.00	....	120.00	....	....	....	....	12,000	119	....
Otero Irr. Dist	10.00	....	35.00	0.60	....	40.00	....	16,000	40	....
Palisades I. D.†	....	....	40.00	....	40.00	40.00	....	6,000	41	....
Paradox Valley	....	....	45.00	....	50.00	....	....	30,000]	45	....
Pueblo Rocky Ford	....	....	75.00	....	75.00	40.00	1.60	50,000	....	150
Redlands I. Power Co.	50.00	....	75.00	....	100.00	....	....	5,000	100	....
Routt Co. Ditch Co.	35.00	....	35.00	....	45.00	....	....	37,000	45	....
South Palisade†	....	....	127.00	....	....	....	....	700	127	....
Denver Res. & Irr. Co.	40.00	....	40.00	....	45.00	....	....	65,000	45	....
Average cost			43.00							
Average cost by U.S.R.S.	59.00.									
CAREY ACTS PROJECTS IN COLORADO										
Great Northern Irrigation & Power Co.	....	....	35.00	....	....	....	....	150,000	35	....
Colorado Realty & Security Co.	....	....	45.00	....	....	....	....	45,875	45	....
Toltec Canal Co.	....	....	40.00	....	....	....	....	16,000	40	....
Colorado Land & Water Co.	....	....	45.00	....	....	....	....	16,000	45	....
Two-Buttes	30.00	....	35.00	....	....	....	....	22,000	35	....
Valley Investment Co.	....	....	40.00	....	....	....	....	24,000	60	....
Average cost per acre	$38.00.									

* Includes assessment for improvements. † Pumping plants. Estimate of reclamation service 37.5% too high.

AMITY CANAL—The acreage under the canal is 45,000, and not 80,000. A water right in the Amity system sold originally at $22 per acre, including both land and water, the water charge being $10 per acre. Between 1896 and 1900 the reservoir system was constructed and rights in it sold at $30 per acre. Since that time there has been spent $23.50 per acre for improvements on the system. Prior to 1900 the annual assessment was 23c. per acre. Since that time it has been gradually raised and is now $1. This system is renowned as being one of the most expensive large systems of the state, both in cost and operation.

The cost, therefore, to the land owners was $63.50 for water, and, for both land and water, $75. Land and water may be purchased at this time at from $75 to $150 per acre, according to the improvements.

BEAVER LAND AND IRRIGATION CO.—This system required the purchase of many high-priced old water rights. All of the laterals and a large part of the main canals are pipe lines of concrete or wood stave.

This system is exceedingly well built and the project covers about 7000 acres, and not 20,000 acres. The price of the land with water varied from $125 to $250 dollars per acre, the average price being $150. The land buyers received the land and water rights free from all incumbrances, and, in fact, on many of the tracts, orchards were already set out. Under any circumstances the cost of water could not be said to have exceeded $75 per acre for this very remarkable system, instead of $175, as stated by the Reclamation Service, which amount included both land and water. This system was built within the last five years and no further expenditures will be necessary.

CATLIN CANAL—This canal covers 18,600 acres of land. The original cost of water rights to the land owners was $10 per acre, the cost of construction was $4 per acre. Subsequently, on reorganization, the water was repurchased at $20 per acre, and water rights and other improvements were replaced at $20 per acre. There is no indebtedness or liability. Cost of maintenance is 50c. to 80c. per year. No further expenditures for improvements will be necessary. The total cost to the land owners was $10 per acre. The present value for water rights is $75 per acre.

COLORADO CO-OPERATIVE CO.—This system contains 4600 acres of actually cultivated land, the cost of maintenance and improvements on which, during the past year, was $6600. This was a strictly coöperative company, operating sawmills, stores, etc., and the secretary states that the building of the ditch was a secondary matter, and that he cannot give an estimate of the cost. He gives the present value as $33 per share covering two acres.

EAST PALISADE IRRIGATION DISTRICT—This is a very small district of 645 acres in the vicinity of Palisade, Colo. utilizing a steam pumping plant, the lift being about 50 ft. Naturally, this project is quite costly. From the information obtained it appears that the cost to consumers was $61.50 per acre.

FORT LYON CANAL—This system covers about 100,000 acres, of which 68,000 acres are irrigated. The original cost to consumers was $10 per acre, and of subsequent improvements $15. Some desired improvements will cost an additional $1.58 per acre. The total cost to consumers has been $25, and will probably reach $26.58. The district is bonded at present for $8.50 per acre, which is included in the "cost." The present value of water rights under this canal is $50. The cost of maintenance and operation of this system approximated $1 per acre in 1912, of which 60c. was spent for improvements and betterments.

THE GRAND VALLEY DITCH CO.—This ditch covers 40,000 acres near Grand Junction and is one of the larger and better canals of that region. Water rights sold originally for $8.25 per in., or $4.13 per acre. When the present company was formed it purchased the system from the Travelers' Insurance Co., and placed the value of water rights at $20 per acre. The present selling price is $60 per in., and the users are permitted to water as much land as possible with each inch. The present value of the water is about $30 per acre, as the general duty of the water is two acres per inch.

GREELEY-POUDRE IRRIGATION CO.—This project is in an uncompleted condition. Work has been stopped on account of interstate litigation over the water rights. Of the $5,100,000 of bonds authorized, less than one-half have been disposed of. It is impossible to say what the ultimate cost of this project will be. The writer assumes it to be $40 per acre, basing his estimate on the bonds authorized.

MESA COUNTY IRRIGATION DISTRICT—This system comprises 2600 acres; bonded indebtedness, $188,000. Cost per acre, $72. Water is supplied by a pumping plant, all of the water being lifted about 120 ft. The principal item of expense was a concrete dam across the Grand River, recently built, the cost of which to this district was about $30 per acre.

ORCHARD MESA IRRIGATION DISTRICT—This system comprises 12,000 acres. The bonded indebtedness is $1,075,000, or $90 per acre, which was the original cost. Other expenditures will probably bring the cost to $120 per acre on the basis of about 10,000 acres. This is an expensive pumping project, covering nine miles of power canal through the cañon. One lift is 50 ft. and one 132 ft. The cost of the lateral system and the first three years interest and maintenance are

*State Engineer, Denver, Colo.

included in the above cost. The cost of rights-of-way alone amounted to about $75,000. The adding of interest and maintenance charges since completion has raised the indebtedness to about $155 per acre.

THE OTERO IRRIGATION DISTRICT—This is one of the irrigation districts first formed. Its original cost was $10 per acre for 16,000 acres under the canal. When the irrigation district was formed the old water rights were purchased from users for bonds.

There have been five issues of bonds: (1) $300,000 for the purchase of the old system; (2) $160,000, and (3) $40,000 for the construction of a reservoir; (4) and (5) $250,000 for the rebuilding of the canal system and the construction of permanent concrete and steel structures. At present, there are about 19,000 acres subject to the bond issue. Lands at present unpatented will increase the taxable acreage to probably 22,000 acres. The bonded indebtedness is now $40, and will be reduced to $35 per acre when all the land is subject to taxation. The total cost to land owners to date has been about $35 per acre. No further improvements will be necessary.

PALISADES IRRIGATION DISTRICT—Comprising 6000 acres; bonded indebtedness, $160,000. This is another pumping proposition with a lift of 90 ft. A total of $150,000 was spent by this district and the Mesa County district, jointly, for the construction of a concrete diversion dam across the Grand River, one-half of which sum was borne by this district. The total cost, therefore, was about $235,000, or $39 per acre.

PARADOX VALLEY IRRIGATION CO.—This project is uncompleted, and nothing definite can be said relative to cost. The water rights are offered at $50 per acre.

PUEBLO-ROCKY FORD IRRIGATION CO.—This project of 50,000 acres is at present in the hands of a receiver. The price given in "Engineering News" includes the land. The reported cost per acre is $37.50; future improvements, $1.60 per acre. The present selling price of $125 includes land. The project is bonded at $40 per acre.

REDLANDS IRRIGATION POWER CO.—This system will comprise about 5000 acres. The cost of water has been from $50 to $100 per acre. This is another costly project, all of the water being pumped, part by direct lift to an elevation of 130 ft.; part by electric power to an additional height of 75 ft., the total lift to several tracts being 255 ft. The electrical development in connection with this project is producing a revenue to the company, by sale of electricity to Grand Junction and for other commercial uses.

ROUTT COUNTY DEVELOPMENT CO.—This system is a Carey Act project. The price for water under contract with the state was limited to $35 per acre. This project has not been completed and the money paid by the entrymen is being held by the state.

SOUTH PALISADE HEIGHTS IRRIGATION DISTRICT—This district, comprising 700 acres, is another expensive pumping project, the water being lifted by a producer-gas plant to a height of 400 ft. It is no wonder that under these conditions the cost is $127 per acre.

DENVER RESERVOIR IRRIGATION CO.—This is an incomplete project. Not only does it comprise the land belonging to the company, but it has also made contracts with two irrigation districts to supply them with water, one at $40 per acre. Land with water is offered at $100 per acre. The writer estimates the cost of the water at $40 per acre.

CAREY ACT PROJECTS

THE GREAT NORTHERN IRRIGATION CO.'S project, comprising 150,000 acres (and not "2121 acres" as stated in the article), is being prosecuted with due diligence; but as this is a very extensive project, it will take considerable time before construction can be pushed, awaiting determination of the water supply and other essential data. Systematic stream gagings are being made.

THE COLORADO REALTY & SECURITY project, or ROUTT COUNTY DEVELOPMENT CO., has already been discussed under that title. The maximum price allowed is $45 per acre.

THE TOLTEC CANAL CO. project is permanently abandoned, and the State Land Board will probably cancel the contract within a short time.

THE COLORADO LAND & WATER CO. had their project under construction, and the fact that the work is not now prosecuted is in no way their fault, but is due to delay in the Interior Department in Washington. Segregation was asked in April, 1906, and approved in October, 1907. Subsequently, the approval was withdrawn, required data were furnished, and instructions complied with. Final approval is still pending.

THE TWO BUTTES RESERVOIR system has been completed, but no maintenance charge has been made during the first year. The cost of construction on this project was $30, and the price paid by the settlers was $35 per acre; 2300 acres are now under cultivation.

VALLEY INVESTMENT CO.—The maximum price permitted under their contract with the Land Board is $60 per acre. In conversation with members of the company, the writer finds that it is the intention to sell water rights at from $50 to $55 per acre, and that when seven-eighths of the water rights are sold the management will be turned over to the water users. This company is required to maintain the system at a cost of not more than $1 per acre per annum for a period of 17 years. The first payment of $6 will be cancelled if the land four years. Interest will not begin until the end of three years. The first payment of $6 will be canceled if the land owner puts $6 worth of improvements on the land during the first year of occupancy. Thus, the probable actual cost per acre will be less than $50, with no interest to pay during the first three years, and no payment due on purchase price during the first four years. The cost, therefore, as compared with Reclamation projects, where payments of principal and maintenance charges begin the first year, may be said to be $40 per acre. This is a liberal contract, but the tendency of private projects at present is along these lines in the belief that only actual cultivators should be placed upon the land, and that it is the part of wisdom to encourage substantial improvements by deferring payments to permit the settler to invest his funds in improvements rather than in payment of his indebtedness for the property. This project is held up in the General Land Office.

The only criticism of the individuals intended herein is that they have permitted publications to go forth without first ascertaining all the facts obtainable and carefully analyzing those facts, so arriving at conclusions which are sound and incontrovertible.

It is assumed by the general reading public that a government publication is reliable, and that conclusions and data are obtained and compiled without prejudice or ulterior motives and such publications have great weight and impress the public far more strongly than any other source of information. Mistakes or errors cannot be readily corrected, and he is indeed rash who calls in question the reliability of our Government work. While we acknowledge that the officials are not infallible, there is a reverence which makes criticism almost a sacrilege. This high place accorded official writings makes it all the more necessary that the officials exercise more than ordinary vigilance that nothing incorrect goes forth. Of late years, however, the volume of reports has so increased that much of the work has been done by men younger and less experienced than formerly—men who do not appreciate the gravity of their position and with whom the wish is too often the father of the thought. In the article under consideration, doubtless the desire to show the U. S. Reclamation Service in a favorable light, by excusing failures and mistakes, has led the compiler into the error of making comparisons without going deeply into the data which at first sight appeared to prove what he wished to prove. The higher officials are at fault in not scrutinizing carefully the article before releasing it for publication.

The attention of the writer was especially attracted to the article by a few statements which he at once perceived to be in error. Upon further investigation, so many errors and inconsistencies, both in data and conclusions, were disclosed, that the following statement was prepared in the hope that it would correct in part the harm which the article would undoubtedly do to the engineering profession, and the private irrigation interests of the country.

Before taking the subject in detail, it may be stated by way of general conclusions that the data published in ENGINEERING NEWS of May 15 are, so far as the Colorado projects are concerned, valueless, because:

(1) The costs given on 4 of the 24 private projects are for land with water, and as the price of land may be either more or less than the price of the water, and the price of the water itself may form a large part of the cost of the project, it is impossible to estimate from the figures given the "cost of the irrigation works."

(2) In one case, the cost of the water per "inch" is given without stating how many acres an "inch" serves.

(3) Nine of the projects are uncompleted, some being abandoned entirely, and, in nearly all cases, only a very small percentage of the whole is completed.

(4) Six of the projects include expensive pumping plants with small acreages and, necessarily, high cost of construction and operation.

TABLE II. COST ($) OF CONSTRUCTION AND MAINTENANCE ON SOME OF THE BETTER AND MORE RECENTLY CONSTRUCTED COLORADO IRRIGATION PROJECTS

Projects	Cost of construction	Original sale price	Cost of subsequent improvements	Total cost to users	Cost of maintenance and operation per year	Present value	Bonded indebtedness	Improvements necessary	Area
Twin Lakes Land & Wa. Co.	7.00	10.00	10.00	17.00	....	60.00	0	0	66,640
Holbrook Irr. Dist.	....	10.00	12.50	22.50	0.70	32.50	32.50	0	20,000
Rio Grande Canal	....	9.00	....	9.00	0.30	15.00	....	0	90,000
Monte Vista Canal	....	7.00	....	7.00	0.15	15.00	....	0	20,000
Fort Morgan Irr. Dist.	....	10.00	14.50	24.50	....	50.00	14.50	0	12,300
Riverside Irr. Dist.	....	37.00	....	37.00	....	37.00	37.00	0	29,500
Water Supply & Storage Co.	....	32.22	....	32.22	1.12	37.50	7.60	0	15,000
Rocky Ford High Line	....	7.80	6.70	15.00	0.60	....	....	0.50	28,000
Terrace Irr. Dist.	20.00	20.00	....	20.00	0.50	20.00	20.00	0	34,000
Trinchera Irr. Dist.	17.00	12.00	6.00	20.00	....	13.00	11.40	2.00	45,000
Costilla Estates	14.00	....	....	35.00	0.50	65.00*	....	....	40,000
Montezuma Valley	....	....	....	20.80	....	....	20.80	....	38,800
* Price of land and water.									
Average cost	$19.00	Average for all projects listed in Tables I and II							$33.70

THE TWIN LAKES LAND & WATER CO.—The original cost of construction was $7 per acre. Water rights sold at $10 per acre. Subsequent improvements, including Twin Lakes Reservoir, were sold at $10 per acre, making the original total cost to users $20 per acre. The present market value is $30 for canal water, $20 for Twin Lakes Reservoir water and $10 for Lake Meredith water, making a total of $60. There is no bonded indebtedness and no necessity for further improvements.

HOLBROOK IRRIGATION DISTRICT—This district contains 20,000 acres. Water rights were sold originally at $10 per acre. Subsequent improvements, including reservoir rights, brought the cost to $22.50 per acre. The bonded indebtedness is $32.50 per acre. The apparent discrepancy of $10 between the price at which water was sold and the bonded indebtedness is on account of the purchase by the district of the individual water rights whose value had advanced $10 per acre. This is one of the most efficient systems in the state. The cost of operation and maintenance is about 70c. per acre.

THE RIO GRANDE CANAL—Water rights originally sold at $1280 for 160 acres, carrying with each 100 in. of water. Subsequently, water rights for 160 acres were sold at $1425 for 50 in., or about $9 per acre, as 50 in. now covers 160 acres. In 1912, the cost of maintenance was about 30c. per acre. The present value of water rights is about $15 per acre. Water is rented under this canal at from $2 to $2.50 per in. per annum. A storage reservoir is now under construction, which will cost about $4 per acre.

THE MONTE VISTA CANAL—A 50-in. water right for 160 acres was sold for $1050, of about $6.50 per acre. Both the Rio Grande and Monte Vista canals are finished systems, and the cost of maintenance in 1912 on the Monte Vista canal was about 15c. per acre.

FORT MORGAN IRRIGATION DISTRICT—This district comprises 12,300 acres. The reservoirs and canals are completed. The bonded indebtedness is $178,600, or about $14.50 per acre. Maintenance and interest charges were $1.60 per acre in 1912. There should be added to this bonded indebtedness, to obtain the total cost to land owners, $10 for original constructions of the old Fort Morgan Canal. Water rights under this canal called for 1.44 cu.ft. per sec. for 80 acres. The present value of ditch and reservoir rights is $50 per acre.

RIVERSIDE IRRIGATION DISTRICT—This district serves 29,500 acres. The project is completed. The bonded indebtedness is $1,047,500, or $35.50 per acre. There are, however, about 37,000 acres in the district. The land not now served can be served through the same system, if the water supply for that acreage proves to be sufficient, thus reducing the cost of the same materially.

WATER SUPPLY & STORAGE CO.—The cost of water to users was $32.22 per acre, of which $7.50 is covered by the bonded indebtedness. The system is completed. Present value of water rights for 80 acres is $3000. Maintenance and operation is $1.12 per acre. A large part of the water for this project is brought across the Continental Divide and from the Laramie River through expensive mountain canals. A large part of the cost of maintenance is on this mountain section.

ROCKY FORD HIGH LINE CANAL—The par value of each share was $100, covering a water right for ten acres, though, in practice, such a water right covers nearly 13 acres. The shares were sold at par, or in exchange for work. Subsequent betterments and purchase of early water rights amounted to $188,000, or about $6.70 per acre. Further improvements contemplated, 50c. per acre. The total cost, therefore, will approximate $15 per acre. The annual assessments range from $6 to $8 per share, or about 60c. per acre. The bonds of the district are being paid off one-tenth annually.

THE TERRACE IRRIGATION DISTRICT—This system comprises 34,600 acres. Bonds authorized, $750,000; sold, $625,000. The sale of these bonds has paid for the construction of the system, consisting of two reservoirs with ditch and lateral systems, one year's interest, one year's maintenance, and the entire interest cost on capital invested during the period of construction covering a period of eight years. The cost to the land owners will not exceed $20 per acre. The project was completed in 1913.

THE TRINCHERA IRRIGATION DISTRICT—This district is located in the San Luis Valley and comprises 57,000 acres of land, of which from 40,000 to 50,000 acres will be irrigated, the amount being dependent upon the water-supply. The cost has been in the neighborhood of $925,000, and the total ultimate cost to water users will probably not exceed $20. This project was built within the last few years and is not yet completed.

THE COSTILLA ESTATES PROJECT—This project comprises 40,000 acres. Under it land and water rights are sold for about $65 per acre. The value of each being about equal, makes the cost of water $35 per acre. The president of the company writes as follows:

"Costilla Estates Development Co. Completed unit to serve about 65,000 acres. Entire cost of construction, purchasing of water rights, upkeep and maintenance for a period of four years, charged against the completed unit alone, $16 per acre. Much of this cost is chargeable to the entire system, and the best estimates obtainable show a probable final construction cost of not to exceed $14 per acre."

The construction, most elaborate and thorough in the state, is equal in permanence and workmanship to any of the Government projects. In the letter of acceptance of the project by C. W. Comstock, former State Engineer, the system is commended as being the best constructed in the state.

THE MONTEZUMA VALLEY IRRIGATION DISTRICT—This project was started in 1885. Two issues of bonds have been made: The first on the formation of the district, May 1, 1907, of $795,000; the second, Dec. 1, 1910, for $125,000; $805,000 of the bonds are outstanding; 38,600 acres are subject to the bonds. There are 60,000 acres under the project. It has two reservoirs with a capacity of 17,275 acre-ft. The cost per acre is represented by the bonds sold of nearly $21 per acre. All of the improvement work has been done since 1907.

(5) The average of cost appears to have been computed by adding the unit cost per acre on each project and dividing by the number of projects, instead of first ascertaining the cost of each project by multiplying the acreage by the price per acre on each project, adding the products and dividing the aggregate by the total acreage of all the projects.

In preparing this article data have been obtained by correspondence with the officers of the irrigation companies and other well informed individuals and from published records. These data have been, almost without exception, corroborated from more than one source of information, and are believed to be as correct as can be obtained without the expenditure of an unjustifiable amount of time, money and labor. The writer has consumed more than a month in obtaining the data.

Costs of Colorado Irrigation Projects as Collected by the Author Compared with Reclamation Service Figures

This discussion is limited [primarily] to Colorado projects, as the writer is not prepared at this time to pass upon the projects in other states, but it is to be hoped that others will take up the projects in their own states, in order that the profession may have reasonably accurate information on the subject discussed and may be able to draw approximately correct conclusions on this class of work which involves an expenditure, on the listed projects only, of an amount of money equal to that of the Panama Canal.

Table I, with its accompanying notes, deals with each of the 24 Colorado projects mentioned in the aforesaid article of May 15. The last two columns show the prices as given in the article in Engineering News from data furnished by the U. S. Reclamation Service and may be readily compared with the corrected amounts, as shown in Column 3. Column 5 shows the price at which water rights can be bought at the present time. The present values are dependent not on the cost, but on the supply and demand. Under most canals the supply of water is not sufficient for all the land thereunder and to purchase a water-right means it must either be taken from land already improved, involving the destruction of the value

of the improvements and of the value of assemblage, or that the transfer of a part of the water-supply to other land will necessitate a more rigid economy, a greater exercise of use and application, and an increased danger of shortage in years of drought. The selling prices, therefore, are generally prohibitive and bear little relation either to cost or what a purchaser could afford to pay; in fact, in the compilation of the table the present selling price was most difficult to obtain, as in many cases there had been no sales for some years and there was no water for sale.

On the Amity, Catlin, Fort Lyon and Holbrook canals the original water-rights were for 1.44 cu.ft. per sec. for 80 acres, while on the Government canals, and other canals now being built, the amount of water estimated as necessary for 80 acres is generally from 0.8 to 1 cu.ft. per sec. The excessive allowance of 1.44 cu.ft. has enabled the owners to irrigate more land, and the original costs given should, therefore, be considerably reduced.

The notes to Table I show conclusively that the original cost of projects, perhaps twenty years ago, averaged about $10 per acre. Subsequent improvements, enlarging construction and water-rights, etc., have brought this cost up to an average of perhaps $25. Even the expensive pumping projects listed furnish water at from $40 to $127 per acre.

Carey Act Projects—The list of the "Carey Act" projects given in the article appears to be correct as to the maximum cost which the builders will be permitted to charge the settlers. The public land will cost the settler 50c. per acre. Each case, however, is somewhat different from the others, as may be seen from the notes on the "Carey Act" projects given after the other notes to Table I.

Costs of Recently Constructed Colorado Irrigation Projects

It appears pertinent in this connection to mention some of the more recently constructed Colorado projects. The writer, therefore, submits a list (Table II) of projects of recent construction which he believes to be comparable with the Reclamation projects. He has been consulting and constructing engineer on several of these, and knows from personal experience whereof he speaks.

Some General Considerations

One great defect, both in government and private enterprises, has been that the settler has exhausted his resources in the purchase of land in the beginning, and has had no funds for improvements of his own.

On the North Platte government [Reclamation Service] project, with which the writer was connected for a number of years, those who filed on the land sold their relinquishments before the time the water was available for the land. The price per acre for the relinquishment covering desirable land had risen as high as $40 per acre and the average was probably $2400 for an 80-acre relinquishment. In the purchase of such relinquishments the terms were all cash and the settlers' resources were oftentimes entirely exhausted in the purchase.

The tabulations show the "Carey Act" projects to be the more expensive by about $10 per acre, as compared with the private projects listed in Table I, and $20 per acre as compared with the private projects in Table II.

A considerable part of the difference is due to the cost of securing the segregations by the Interior Department. An expenditure of $10,000 is not uncommon in getting the filings through the several departments of the Government. For instance, the cost of securing right-of-way for a portion of the reservoir site of the Terrace Irrigation District was about $10,000. This sum was spent for attorneys' fees and their expenses traveling to Washington, and in the employment of special agents to see that the papers and documents were not lost or pigeon-holed in the departments.

The delay in securing approval of segregations discourages investors, and delays the completion of the work, increases interest charges during construction, and adds a burden entirely unnecessary and of no benefit to anyone, but materially increases the cost.

The average cost of all of the Colorado projects listed herein is $33.70 per acre; but when it is considered that the prices given include contemplated or needed future expenditures and, in most cases, interest and maintenance charges for from one to four years, also, a disproportionate number of pumping plants of unusually large cost, together with some projects abandoned on account of the high cost per acre, and in the case of the "Carey Act" projects, the maximum price which can be charged, it is certain that the $33.70 is much too high an average cost for the projects listed—in fact, very much too high for average cost of projects at this time, as shown by Table III.

There are some 9000 canals in Colorado. Of the 35 listed herein, all are of comparatively recent construction, and 22 were built or started since the Reclamation Act went into effect, and nearly all of the improvements on the other 13 were made since 1902—that is to say, the work has been done during the same period and under the same conditions of cost of labor and material as have the Reclamation projects.

It is the judgment of the writer that the cost of these projects, on the same basis as the Reclamation projects, would not have exceeded $30 per acre. In fact, it is his experience that a cost of more than $30 per acre is justified only under exceptionally favorable conditions of soil, climate, markets and settlement.

The average cost of private projects built in Colorado since 1902 does not exceed $25 per acre, and the average price charged for water per acre does not exceed $35 per acre. The price for both land and water on new projects does not average over $50 per acre. The cost of water alone on Government projects in Colorado appears to be in excess of $75 per acre, as shown by the Government estimates, and involves in addition a necessary residence of five years on the land, a limited area of ownership, an inability to borrow money to make improvements, a high maintenance charge (on the Uncompahgre project $2.24 for 1911), added to the uncertainty as to actual ultimate cost and date of completion.

The cost of maintenance on the private projects listed, other than the pumping projects, varies from 15c. to $1.12, the average being about 50c. per acre. It is the writer's judgment that on the larger, more complicated, best managed and most satisfactory canals, the cost of operation and maintenance is seldom more than $1, the average on such canals being probably less than 75c. per acre per annum.

It is his experience, also, that the cheapest and most efficient management is that under the direction of the

farmers themselves. The cost is lowest on the individual and community ditches, is higher on canals operated by an irrigation company, and highest on the Government works, although the Government works are supposed to be better built, more permanent and, consequently, should be cheaper.

This is explainable by the fact that the farmer, repairing his own canals, does not count his time; he does not demand as much of himself and, more than all, he is wide awake to prevent or stop breaks promptly. The corporation or company, on the other hand, must pay cash for its labor; it is expected to run on schedule, to repair breaks at once regardless of cost, and is unable to rely on local efficient help.

The Government, in addition to the trials of a company, has a vast amount of "red tape"; it maintains a permanent force comparable to the fire department of a city, a heavy overhead charge, and, above all, is considered legitimate source of ready money and easy work. It is the writer's opinion that the Government has thoroughly spoiled the water users by its paternalism and assumption of burdens properly belonging to the users, who could maintain and operate their lateral ditches, at least, both cheaper and more satisfactorily to themselves than can the Government.

The experience of canal companies during the past thirty years has been such that few will now agree to maintain and operate the canals after a majority of the water rights are sold. The Government and its water users have this principle of economy still to learn.

Reclamation Service Projects

Table III was compiled from the Eleventh Annual Report of the U. S. Reclamation Service, the Reclamation Record, and the report of the Board of Army Engineers.

It appears therefrom that the cost of the Reclamation projects, as at present estimated, is $52. However, in the column giving the Army estimate several projects are estimated too low, and in the column showing the cost as fixed for repayment by the Reclamation Service, also, several appear to be too low and for a number of the more expensive projects the cost to the settlers has not been fixed.

Commenting on the several projects, it appears from information derived from sources other than the U. S. Reclamation Service reports that the estimate of the cost of Salt River, Yuma, Truckee-Carson, Uncompahgre, Grand Valley and Klamath projects is too low. The writer makes the following estimate as his conclusion of the lowest cost for these projects to settlers, assuming that all expenditures thereon are to be returned to the U. S. Reclamation Service:

Salt River	$65 per acre	
Yuma	100 "	
Grand Valley	100 "	
Uncompahgre	100 "	If reservoir water is supplied
Truckee-Carson	50 "	

Increase due to worthlessness of a large part of the land.

As the water-supply is questionable on the Milk River, Sun River and Shoshone projects, a reduced acreage will necessitate increased charges per acre on these projects. The Army engineers say, also (p. 18):

> A positive loss will occur on Lower Yellowstone project even under the most favorable condition of completion, and in the Missouri River and Garden City Pumping projects, and in the Hondo Project large losses appear to be inevitable.

On the Sunnyside unit of 100,000 acres, 12,000 must be pumped on at an additional cost above the $52 given. The area under all the projects (p. 11, Eleventh Annual Report U. S. R. S.) is stated to be 3,010,689 acres; the area for which water is available during 1912, 1,158,830 acres; and the area irrigated (p. 292) 564,681 acres.

The cost per acre on the projects seems to constantly increase, due in part to a decreasing acreage, and in part to increased total cost.

It must also be remembered that there will be uncollectable charges, individual failures, land will prove to be valueless, all of which causes will be either a loss to the Reclamation fund or an added burden to the settlers; also, 3% on $10,000,000 for ten years (or $3,000,000) (p. 17, Army Report) must be paid by someone. Page 11 of the Army Report says:

> In addition there was to June 10, 1910, an additional revenue of $2,086,174 derived from sale of town lots, sales of water, leases of power, etc., which are under the law credited as a reduction of the cost of the project from which the receipts are derived.

These considerations lead the writer to the conclu-

TABLE III. COST PER ACRE OF U. S. RECLAMATION PROJECTS COMPILED FROM GOVERNMENT RECORDS

Project	Per cent. complete	Estimated cost of construction per acre in dollars: By Army Board	By U.S.R.S.	Remarks by Board of Army Engineers
Salt River	93.7	45.00—55.00	.....	Area depends on water-supply.
Yuma	73.7	60.00—73.00	66.00	
Orland	84.	45.00	48.00*	25% must be added if project is extended. Partly irrigated by private ditches.
Grand Valley	8.	88.50	.....	
Uncompahgre	54.9	70.00	70.00*	Present irrigation largely from private ditches.
Minidoka	90	22.00—36.30	30.00—40.00	
Boise	52	66.00	60.00	
Garden City	98	40.00	37.50	Will entail loss to Government. Part of land irrigated with flood water.
Huntley	96	30.00—36.60	30.00—36.00	
Milk River	12	39.00	.....	Area dependent on water-supply irrigated at present with flood water.
Sun River	9	35.00	47.00*	Area may be largely reduced account of water-supply.
Lower Yellowstone	95	57.00	45.00	Will entail large loss to Government.
North Platte Interstate	82	45.00	55.00	
Laramie	0	50.00	.....	
Goshen Park	0	80.00	.....	Area indefinite.
Truckee Carson	69	35.00—40.00	22.00—36.00	Area dependent on water-supply, may be more than $40.00 per acre.
Carlsbad	100	31.00	45.00	
Hondo	100	44.00—50.00	45.00	Will entail a loss to Government. Small area irrigated by flood water.
Rio Grande	20	36.00	.....	
Missouri River Buford	50	51.70	38.00†	Will entail a loss to the Government.
Missouri River Williston				
Umatilla	80.7	60.00—100.00	70.00‡	Does not include proposed extension.
Klamath	71	30.00—63.33	30.00	
Belle Fourche	93	30.00—35.00	30.00—40.00	
Strawberry	62	56.80	.....	Water-supply uncertain.
Okanogon	100	68.30	110.00	
Yakima Sunnyside	88	52.00	52.00	
Tieton	94	93.00	93.00	In part under flood irrigation.
Shoshone	50	50.00	50.00—52.00	Area may be largely reduced account water-supply.
Kittitas	.....	75.00	.....	

Average cost per acre about $52.00

* Estimated on basis of expenditures, area and per cent. completed. † Cost on basis of cost, percentage of completion and area appears to be $68. ‡ Cost to Government about $74 on basis of area, percentage completed and cost.

tion that the average cost will be in excess of that shown in the tabulation, and that the cost now estimated as about $52 will rise to an average of $60 or more before the projects are completed.

The cost of maintenance, as shown by the Eleventh Annual Report of the Reclamation Service (pp. 292-293) appears to be $1 and the cost of operation 88c., or $1.88 for both.

A considerable part of the construction cost (see footnote, p. 18, Eleventh Annual Report U. S. R. S.) may be due to the addition of maintenance charges to the construction cost, but it is also the custom in private enterprises for the company to pay maintenance charges until the lands are sold. Therefore, if such maintenance charges are included in the Government's construction charges the figures are comparable, but if such are not included, then the costs shown for private enterprises should be reduced somewhat before a comparison is made.

The total expenditure by the Government to date is for construction $72,042,173, and the area ready to be served 1,158,530 acres (about $62 per acre). The last census gives the cost to July 1, 1910, as $67.52.

On the projects the easier portions were built first. Much of the land was already irrigated more or less adequately, and already had more or less satisfactory irrigation systems, the amount, as near as the writer can judge, being about one-third of that now ready to be served, or 400,000 acres.

The structures built first, however, were designed for the completed systems. This procedure, therefore, will reduce the cost of reaching the lands not yet reached, so that, although the portions finished at this time were generally the easiest covered (much of the area being already supplied with water), the cost of covering the (approximately) 1,800,000 acres not now furnished with water should not exceed the cost per acre of the land now covered ($62).

The ultimate cost of irrigating 3,000,000 acres should not exceed $180,000,000; of this 3,000,000 acres, however, probably no more than 2,500,000 (four times the present irrigated area) will within any reasonable future time be covered by the present projects, the expenditure on which should be little, if any, over $150,000,000.

It should be noted that only about one-fifth of the area in the projects is irrigated ten years after construction began, and that on the projects which are more than 90% completed, the following areas under the projects and the areas irrigated in 1912 are shown by the Government reports:

Project	Total acres under project	Acres irrigated	Percentage
Salt River	230,000	115,514	50
Garden City	10,670	0,000	00
Huntley	32,405	12,000	37
Minindoka	118,700	71,000	60
Lower Yellowstone	60,116	15,443	26
Carlsbad	20,277	14,803	74
Hondo	10,000	1,135	9
Belle Fourche	100,000	19,800	20
Okanogan	9,900	6,467	64
	592,068	256,162	43.2

Causes of High Cost of Government Projects

To one familiar with Government work the causes of the necessarily high cost are obvious:

(1) Public money is being used.

(2) Contractors are loath to bid on Government work, as many who have done so have been financially ruined. They are required to do much unnecessary "finishing, polishing and sandpapering," and the inability of the contractor to force the United States into court for a judicial determination of matters in controversy, especially small ones, renders his position precarious.

(3) The high overhead consulting and collateral charges, and charges for investigating other projects found not to be feasible.

(4) The eight-hour day and rigid exaction of penalties for nonfulfilment of contract.

(5) The difficulty in substituting material or work equally good when to obtain that specified (while perhaps not impossible) is impracticable, costly and inconvenient to the contractor.

(6) The practical absence of discretion on the part of the engineer, relative to changes or interpretation of the contracts.

(7) The delays and the "red tape."

All these things make it necessary for the contractor to bid "safe."

Moreover, the fact that the Government has had "plenty of money" has led it to build monuments rather than structures of necessary dimensions—where one can disregard first cost there is danger of ill-advised expenditure.

On the basis of $60 per acre, average cost, and $1.88, average maintenance for ten years, the average cost would be $78.80, which, with 3% interest on the $20,000,000 loan, would add $3,000,000, bringing up the total average cost to at least $80 per acre.

The cost to consumers on all the private Colorado projects herein averages $33.70. The interest cost for ten years, at 6% on deferred payments, would add about $10; maintenance, operation and betterments, $7, a total cost of $50.70 per acre.

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Advantages and Disadvantages of Government and of Private Irrigation Projects

The advantages on the Government projects are:

(1) No interest charges excepting on the $20,000,000 loan by the Government.
(2) Better construction and certainty of the ability of the Government to finance the project to completion.

The advantages on private projects are:

(1) Lower first and total costs.
(2) Lower maintenance charges.
(3) Earlier control by the users.
(4) Earlier completion of the project and enforceable contracts.
(5) Power to buy and sell at will.
(6) Power to borrow money to make improvements.
(7) The right to reside and work elsewhere while developing the land.

It might be contended that on the Government projects the lands are free, but this is not true. Nearly 60% of the land is privately owned; therefore, the same must be bought, and the ultimate owner on Government land must, almost without exception, purchase a relinquishment.

The disadvantages of private enterprises are:

(1) Uncertainty of financing.
(2) Profits and interest paid builders.
(3) The tendency toward insufficient investigation before construction, and the general hurry all along the line.

The disadvantages of Government projects are:

(1) The settler has no enforceable contract with the Government.
(2) The uncertainty of ultimate cost.
(3) Necessary residence on the land.
(4) Limited acreage, which eliminates all possible compensation in the form of profits by speculation, which profits are necessary as an inducement to get people to "pioneer" and suffer hardship, break away from old associations and take chances in a new country under new conditions.

The time, we are glad to say, has not yet arrived in the United States when people are satisfied merely to make a living without hope of ultimate independence in old age and a sufficiency to enable them to properly raise and educate their children. This lack of inducement accounts in large measure for the slowness of settlement and cultivation of lands under Government projects.

Cost of Irrigation Projects by Ownership or Class and by States

An examination of the Thirteenth U. S. Census Report in regard to irrigation is interesting, and it is suggested that the reader examine the figures set forth therein. It appears (p. 423 "Abstracts with Supplement for Colorado") that the increased acreage between 1899 and 1909 was 1,180,761 acres, while between 1899 and 1910 the increased investment in irrigation works was $44,877,740. Deducting the probable expenditures for 1910 the increased investment 1899 to 1909 would be about $40,000,000, or a little over $33 per acre, which figure agrees very closely with the estimates given herein on private projects.

Tables IV and V, taken from the Census Report, are self-explanatory. Table IV shows that the "Carey Act" enterprises cost less than one-half that of the Reclamation Service, and that commercial enterprises cost a little over one-third as much as did the Reclamation Service projects. Table V shows the cost in the several states, and the expense of maintenance and operation. In this table the cost of the Government enterprises is included and raises the average somewhat, both as to cost and as to maintenance and operation.

Final Conclusions

(1) The Government enterprises cost 25 to 50% more than private enterprises, after making due allowance for interest charges and maintenance.

(2) Maintenance under Government projects is more than twice that under private projects.

(3) The original cost of private enterprises is about one-half that of Government enterprises.

(4) As a general proposition, ownership of land under private irrigation systems is the more advantageous to the farmer, aside from considerations of cost.

TABLE IV. COST OF VARIOUS CLASSES OF IRRIGATION ENTERPRISES

Taken from the Thirteenth U. S. Census Report

Class of enterprises	Average cost per acre based on: Cost to July 1, 1910, and acreage enterprises were capable of irrigating in 1910	Cost to July 1, 1910, and acreage irrigated in 1909	Estimated final cost and acreage included in projects
All classes	$15.92	$22.41	$13.64
U.S. Reclamation Service	67.52	134.17	48.14
U.S. Indian Service	12.78	27.83	13.33
Carey act enterprises	30.53	115.30	21.75
Irrigation districts	27.37	41.44	20.33
Coöperative enterprises	12.89	17.19	10.07
Individual and partnership enterprises	7.09	8.69	5.22
Commercial enterprises	24.08	41.71	10.79

TABLE V. COST OF IRRIGATION ENTERPRISES BY STATES

From the Thirteenth U.S. Census Report

	1910			1909
	Cost to July 1, 1910, and acreage enterprises were capable irrigating in 1910	Cost to July 1, 1910, and acreage irrigated in 1909	Estimated final cost and acreage included in projects	Maintenance and operation. Average per acre for which cost was reported
Total	$15.92	$22.41	$13.04	$1.07
Arizona	45.60	55.23	26.30	0.93
California	20.05	27.24	15.37	1.54
Colorado	14.19	20.29	12.92	0.75
Idaho	17.15	28.64	16.47	0.63
Kansas	9.75	36.44	8.47	1.59
Montana	10.42	13.68	9.21	0.89
Nebraska	18.17	30.47	13.95	1.09
Nevada	7.99	9.58	9.89	0.97
New Mexico	14.19	19.83	10.56	1.36
North Dakota	38.17	81.62	21.91	28.40
Oklahoma	7.38	10.76	5.53	0.51
Oregon	15.36	18.00	15.52	0.75
South Dakota	23.69	43.11	18.85	0.64
Texas	21.57	44.72	11.43	3.25
Utah	11.22	14.04	9.16	0.65
Washington	34.47	48.51	27.32	3.08
Wyoming	10.80	15.62	9.18	0.86

(5) Private capital can finance and build any irrigation system that the Government would be justified in undertaking, under the assumption that the Government expenditures are to be repaid.

[Since Mr. Field's communication questions the accuracy of figures sent out by the U. S. Reclamation Service we have given the Service opportunity to reply to it in this issue, in order that communication and reply may appear together.

Mr. Newell states, in effect, that the figures questioned by Mr. Field were sent out tentatively, and were subject to revision after the receipt of corrections and criticisms which were invited. We did not understand that the figures were tentative when we prepared our article for publication, nor did we anticipate the part we were thus to play in eliciting suggested corrections (besides Mr. Field's criticisms of the Colorado figures, above, see also letter from H. G. Watson on p. 128 of our issue of July 17, 1913, criticizing some of the Wyoming figures in the same article). Mr. Newell's reply follows.—Ed.]

Sir—I have read with interest the advance proof of the article by John E. Field on the cost of the Reclamation Service and other irrigation projects in Colorado. Mr. Field has brought together many important figures and drawn conclusions with which most of us can agree in part at least.

The figures in question, as published in your issue of May 15, 1913, were evidently taken from a circular letter sent out from the office of the Reclamation Service on March 17, 1913, "giving a partial list of various private works under construction or recently completed with the approximate rate at which water is sold per acre," with the request, "please make suggestions or criticisms of the statements and inferences." This circular letter was sent to many of the managers of the larger irrigation works and replies have been received verifying or correcting the figures given. Probably the most elaborate discussion brought out thus far is this one by Mr. Field, who falls into the error of assuming that these figures are final.

It is hoped that others who are interested in the subject will follow Mr. Field's suggestion to criticize these figures and give statements which may be considered authoritative.

These comparisons of cost of irrigation works are as difficult and elusive as those of the cost of any complicated system, such as a railroad or water-works, and it is appreciated that valuable comparisons can be made only by one man or body of men following the same method for each of the systems under comparison. Especially is this the case when attempting to go into considerable refinement, or to consider not merely the original cost but to distribute this over the output or product, such for example, as the cost per acre, as we then have a double set of figures and inferences: first, as to the actual outlay, and second, as to the net area over which this outlay is to be distributed. If, for example, water-rights are purchased at the rate of $20 per acre and these rights are found to be so limited, from the fact that water obtained for three acres is barely sufficient for one, then the estimated cost of $20 per acre should really be corrected on the basis of the output and become more nearly $60 per acre.

It appears that Mr. Field overlooked some of the qualifying footnotes to Table I of your article (p. 1008, May

15, 1913); or at least did not reproduce them in making up his own Table I. The $175 per acre for the Beaver project, Table I, is evidently a typographical error [in our original article—ED.] as the original figure was $75 instead of $175. For such reasons, the force of some of Mr. Field's conclusions evidently fail.

The main point of the article, however, is assumed to be summed up in the conclusions rather than the figures. By making selection of certain groups of canal systems built at certain times under certain conditions, it is possible to arrive at almost any inference. As a matter of fact it is well known that most of the easily available sources of water-supply had been taken up when the Reclamation Act, June 17, 1902, was enacted and that the choice of projects was limited largely to those which could not readily be undertaken by private capital, or where there were certain legal or interstate complications, such that apparently the Government alone could undertake the work. Thus a comparison of actual cost of Government works with those built by private parties at the same time should take into consideration many of these matters as well as to whether adequate water storage had been provided.

The cost of the Uncompahgre Project with its long and expensive tunnel and proposed extensive reservoir system can hardly be compared with the average project without tunnels and with little if any water storage. As to whether the Government was justified in building such an expensive work or not is, of course, another side of the case. There is no doubt but that many projects have been built in Colorado at far less cost than others can be built either by private or public capital.

Taking up Mr. Field's final conclusions:

(1) "That Government enterprises cost 25% to 50% more than private enterprises, after making due allowance for interest charges and maintenance." This, of course, depends upon what is included by private enterprises. It is possible to show that Government works, which are limited in number, and especially difficult features, have cost far more than this ratio when compared with some of the older irrigation systems. If the Government could or would confine its work to the easier projects built without long expensive main canals, or tunnels, and reservoirs, or if works of similar difficulty were compared on the basis of similar water-supply actually available, it is believed that this conclusion would not stand.

(2) "Maintenance on the Government projects is more than twice that under private projects." This also depends upon the comparison made. Taking private projects in similar degree of development and with similar difficulties, the difference is not as apparent.

(3) "The original cost of private enterprises is about one-half that of Government enterprises." This is true as it stands, but as above stated a fairer comparison would be of recently built private works comparable in size and difficulty with those built by the Government.

(4) "As a general proposition, ownership of land under private irrigation systems is the more advantageous to the farmer aside from considerations of cost." This is yet to be demonstrated. There are, it is true, certain disadvantages inherent with the entry and settlement of public lands and the securing of the return to the Government of the investments already made, arising largely from the fact that the conditions imposed by law are novel and have not yet become thoroughly understood by the public in general. It is significant, however, that whatever criticisms are made of these disadvantages the propositions to relieve them are often vigorously opposed by the farmers themselves.

(5) "Private capital can finance and build any irrigation system which the Government would be justified in undertaking under the assumption that the Government's expenditures are to be repaid." This is an inference which leaps far beyond the data quoted. It is difficult to follow the line of reasoning largely because of the fact that private capital did not or could not finance and build the irrigation systems undertaken by the Government. The inference from this conclusion is that the Government should not have built these particular works. It is very easy to make such criticism, as the matter is one of judgment based upon incomplete information, the discussion of which would require more time than is available. Taking, however, the projects which have been built in whole or in part by the Government, it is difficult to imagine how any one of them could have been financed and built under the conditions which surrounded them at the time that the Government took up the work.

(6) "The Government would be justified in building projects which will not repay the total expenditure by reason of the indirect benefit to the country as a whole." This is presumably the crucial point of the argument by Mr. Field, namely, that while the Government work may be more expensive yet it is justified by the gain to the entire community. However this may be, it is obviously the duty of everyone connected with the Government work to keep down expenditures as far as possible and secure the highest degree of efficiency and economy in the operations. This is and has been the guiding principle of all of the responsible men in charge of the work. Every effort is being made in this direction and while carefully observing all of the laws and regulations imposed upon Government work, the operations themselves are being modified and improved from day to day toward this end.

F. H. NEWELL,
Director U. S. Reclamation Service.

Washington, D. C., July 30, 1913.

The Marin Municipal Water District, with headquarters at San Rafael, Calif., is the first water district to be formed under a new law in California, which permits small incorporated towns and communities to combine for the purpose either of acquiring present water-supply systems or building new ones, for the benefit of water consumers within the district. Among the 13 directors of the district is M. M. O'Shaughnessy (presumably the present City Engineer of San Francisco). R. H. Trumbull is president, S. M. Augustine is general manager, and A. R. Baker is engineer of the district. We are indebted to the latter for the following statement regarding a report on the preliminary study of possible water-supplies available for the district:

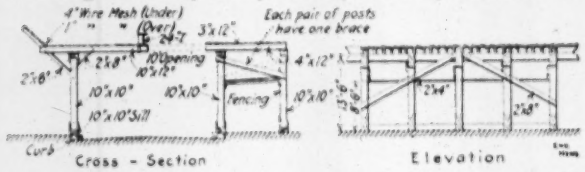
Although several schemes were presented the most logical one and the one most likely to be used is the development of the Lagunitas Creek watershed. The first installation calls for the delivery of 5,000,000 gal. of water daily, developed through the building of the proposed Alpine Dam, of concrete, 125 ft. high, and storing 2,500,000,000 gal., the Pine Mountain tunnel, 8730 ft. long and connecting conduit flow and pressure lines, and the purchase of the present two operating companies, the Marin Water & Power Co. and the North Coast Water Co., together with 5300 acres of watershed in land from the Howard-Shafter Estate. The cost of this first installation is estimated at $2,000,000. To increase the daily supply to 15,000,000 gal. the Alpine Dam will be raised to a total height of 170 ft. when it will store 6,300,000,000 gal. and the necessary additional conduit lines will be added. The estimated cost of this ultimate system will be $2,882,000.

The Lagunitas watershed lies just north of Mt. Tamalpias, between what are known as the Bolinas and Pine Mountain ridges, and the steep rocky and absolutely uninhabited slopes, together with the phenomonal (for this section of the country) average annual rainfall of 60 in. make it not only wonderfully productive, but ideal from a sanitary standpoint.

Protecting Street Traffic from Falling Objects during Building Construction

An unusually efficient protection to street traffic against danger from falling material during the course of building construction, has been provided in front of the building which is being erected for the Western Union Telegraph Co., at 18-20 Dey Street, New York City. The building laws require that where construction work is in progress, a bridge shall be erected over the sidewalk, for the protection of pedestrians. The contractors, in this case, have not only done this, but have, in addition, covered the whole street and the opposite sidewalk as well, with a combination of wooden planking, steel cables and wire screens.

As shown in the accompanying sketch, the portion over the street and sidewalk, adjacent to the building under construction is covered with wooden planking 3x10 in., supported on 4x12-in. wooden beams. This bridge, which is 13 ft. 6 in. above the sidewalk, furnishes space for receiving and temporarily storing building material. A



Cross-Section of Overhead Screen Protecting Street Traffic During Construction of Western Union Telegraph Building

corrugated iron shield placed 5 ft. below the platform protects pedestrians from water dripping from the platform, in the event of rain.

An opening, 30x10 ft., has been left in the portion of the street-covering adjoining the wooden platform over the sidewalk. Through this opening, material can be passed up to the platform which bounds one of the longer sides of the hole. In order to provide more space for handling and temporarily storing material, planks have been placed at each end of the opening. In this way about 400 sq.ft. of additional platform space is made available.

From the edge of the platform and the side of the aperture parallel to the platform, seventeen steel cables, ¾ in. in diameter, spaced about a foot apart, extend longitudinally the entire length of the structure, and are lashed securely to the ends of the wooden framework. The side of the structure, opposite to the new building, ends in a 5-ft., 45-deg. rise, as shown in the section. On this oblique framework of 2x6x-in. wooden cantilever arms, five runs of 1½-in. angle iron have been fastened parallel to and spaced the same as the cables.

Over the cables and angle iron two wire screens have been laid. The upper, which is of expanded metal of 1-in. mesh, rests on the lower screen of ⅜-in. galvanized wires of 4-in. mesh. The cables and screens are supported on 10x12-in. wooden cross-beams spaced about 8 ft. apart. These beams rest on 10x14-in. longitudinal beams, supported by 10x10-in. wooden columns, except at the side of the opening, where the cross-beams are suspended, by means of hangers, from a 24-in. I-beam.

Considering the amount of building construction all the time going on in a great city, it is remarkable that more accidents from falling objects do not occur.

There have been, however, enough fatalities in New York City from this cause to prove the value of such a protection as that herewith illustrated.

The special feature of merit in this protection is that it affords great strength to resist the impact of a heavy object falling from a great height, and it does not obstruct the light like the ordinary plank covering. In various cases where heavy pieces have fallen from high buildings during erection, they have crashed through the wooden planking over the sidewalk. The steel cables in the protection herewith shown offer the best safeguard against this danger that we have anywhere seen.

Water and Air Movement in Relation to the Contamination of the Lake Water-Supply of Auburn, N. Y.

A notable study of water and air movements at Lake Owasco, N. Y., has recently been made by J. Walter Ackerman, superintendent of water-works of the city named. This study, with other interesting data, was reviewed by Mr. Ackerman at the convention of the American Water Works Association, in June, 1913. The study was made to secure evidence for use in a lawsuit brought by the city against the Board of Education of Union Free School District No. 1, in the Town of Moravia, N. Y. The sewer from this school of 300 to 400 pupils discharges directly into Mill Creek, one of the tributaries of the main inlet to the lake. The school is some four miles distant from the lake, and the lake is the source of the water-supply of Auburn.

In October and November, 1907, there were some twenty cases of typhoid fever on the drainage area of the lake, mostly in Moravia. In the following spring 45 cases of typhoid fever, with 12 deaths, were reported as occurring in Auburn. The typhoid-death rate for the year was 46.8 per 100,000. Connected with studies of the outbreak and studies for and testimony given in the lawsuit were Profs. O. H. Landreth, Geo. C. Whipple, H. N. Ogden, C.-E. A. Winslow and W. P. Mason. We shall concern ourselves here with Mr. Ackerman's study of air and water movements.* Quoting now portions of Mr. Ackerman's paper:

> Owasco Lake is a long, narrow body of water, its long dimension of about ten miles being approximately north and south; it has a surface area of 10.32 sq.mi., a land watershed of 193.36 sq.mi., and a total capacity of 200,000,000,000 gal. The main inlet, which enters at the south end of the lake, drains over 60% of the watershed area. Practically all of the sewage pollution on the entire watershed comes into the lake through this main inlet from the villages of Moravia, Groton and Locke; the first two having less than two thousand inhabitants, and the latter only a few hundred.
>
> The City of Auburn, N. Y., gets its supply from this lake, through an intake pipe from the opposite (or north) end of the lake, at a point about half a mile from the north shore [and some 14 miles from the sewer outlet in question] at a depth of 40 feet.
>
> Realising that our case would fail unless we could prove [the transmission of infection from the sewer to the intake] some special experiments were undertaken by the writer, upon the suggestions of Professor Whipple. These were to establish, if possible, the fact that the wind blowing over the waters of the lake would create surface currents of suffi-

*The full paper, with discussion by most if not all the men would, and presumably with maps and diagrams, will appear in the Proceedings of the American Water Works Association; J. M. Diven, secretary, Troy, N. Y. Some further particulars may be found in our report of the convention of the American Water Works Association, July 3, 1913.

velocity to bring polluted water from one end of the to the other well within the vigorous lifetime of danger- pathogenic bacteria.

The method of operation was to place some specially de- d submerged floats of varying depths in the waters of sco Lake and note their movement, both as to direc- and velocity, this giving a knowledge of the currents in ake. To compare this with the wind movement, a stand- recording anemometer was installed at the exposed end the State pier in Owasco Lake.

There were five floats constructed. Nos. 1 and 2 were in equilibrium at a depth of 5 ft.; No. 3, 10 ft.; No. 4, 15 ft., and No. 5, 20 ft. The only projection above the water was a slender pole about 6 in. high and less than 1 in. in cross-sectional area. From the top of this was supported a very light tin telltale flag 10 in. square, free to revolve about a vertical axis; and on each, in red and white, was a special symbol to designate each float. At the submerged bottom end of each there were two large vertical sheets of light galvanized metal placed and held at right angles to each other, so that whatever phase was presented to the currents of the water it would be substantially of the same area. Over 99% of the surface was submerged, and over 95% of the surface was at the depths as indicated.

In order to determine the movements of the floats, a base line was established between two prominent hills beside the lake. Observers with watches timed alike were stationed at those two points and angle readings were taken on each float at every 15-minute interval, and they were so planned that an observation was made every three minutes. This gave sufficient data so that the coördinate position of each float was computed to the nearest tenth of a foot for every observation, this of course giving its direction and velocity for any period covered by the experiment. All of these results were plotted. In order to make a comparison of this movement of the waters of the lake with that of the wind, the anemometer charts were tabulated hour by hour, and these results also plotted.

For purposes of determining the rate of the movement of water, as compared with that of the wind, a tabulation (Table I) of "Water and Wind Movement," has been made. As has been noted, the lake is approximately only one-tenth as wide as the length of the same. It was observed that when the wind blew at an angle to the long axis of the lake the resultant movement of the water was generally more nearly parallel to the axis of the lake than to the direction of the wind. And as this axis is practically north and south, a second series of tabulations was made to compare the north and south components of the wind movement and the resultant north and south components of the water movement.

TABLE I. WATER AND WIND MOVEMENTS

Float No.	Time	Travel of floats Ft.	Travel of floats Ft. per hr.	Wind movement Miles per hr.	Wind movement Ft. per hr.	Per cent. which float travel was of wind movement
			Oct. 9, 1911			
1	12:00—2:15	2150	956	6.0	31,680	3.02
2	11:48—2:18	2290	916	6.0	31,680	2.89
3	2:06—4:36	1400	560	6.1	32,208	1.74
4	2:09—4:39	700	280	6.1	32,208	0.87
5	2:12—4:42	600	240	6.1	32,208	0.75
			Oct. 10, 1911			
1	10:30—4:30	7180	1197	7.5	39,600	3.02
2	10:33—4:18	6600	1148	7.5	39,600	2.90
3	10:36—4:21	4500	772	7.5	39,600	1.95
4	10:39—4:24	2600	452	7.5	39,600	1.14
5	10:42—4:27	2080	353	7.5	39,600	0.89
			Oct. 12, 1911			
1	1:15—4:30	3750	1154	17.0	89,760	1.29
2	1:18—4:18	3550	1183	17.0	89,760	1.32
3	1:21—3:51	2750	1100	17.0	89,760	1.23
4	1:09—3:54	2810	1022	17.0	89,760	1.14
5	1:12—3:57	3020	1098	17.0	89,760	1.22
			Oct. 26, 1911			
1	1:00—3:15	1960	871	6.3	33,264	2.62
2	1:03—3:18	2170	964	6.3	33,264	2.90
3	1:06—3:06	1710	855	6.3	33,264	2.57
4	1:09—3:09	1640	820	6.3	33,264	2.47
5	1:12—3:12	1590	795	6.3	33,264	2.39
			Nov. 4, 1911			
1	1:00—4:00	2450	817	10.0	52,800	1.55
2	1:03—3:48	2350	855	10.0	52,800	1.62
3	12:51—3:51	2420	806	10.0	52,800	1.53
4	12:54—3:54	2060	686	10.0	52,800	1.30
5	12:57—3:57	1710	570	10.0	52,800	1.08

Floats 1 and 2 were in equilibrium at a depth of 5 ft.; Float 3, at 10 ft.; Float 4, at 15 ft.; Float 5, at 20 ft.

RELATION BETWEEN WIND MOVEMENT AND TRAVEL OF THE SURFACE WATER

From Table I a plotting was made to give a curve for 5-ft. floats, using wind velocities in miles per hour as abscissas, and the percentage which the water travel was of the wind movement (as shown by an average of the 5-ft. floats) as ordinates. From this curve Table II was deduced.

TABLE II. DEDUCTIONS FROM CURVE FOR 5-FT. FLOATS

Velocity of wind, mi. per hr.	5	10	15	20	25	30
Percentage which water travel was of wind movement	4.25	1.95	1.40	1.15	1.05	1.00
Wind movement in miles required to cause the water to travel 10 miles	307	512	714	869	952	1000
Number of hours of wind at the given velocity required to cause the water to travel 10 miles	61.4	51.2	47.6	43.5	38.1	33.3

Another plotting and curve was made from Table I, using the values obtained by taking the average of the 5- and 20-ft. floats. From this curve Table III was deduced.

TABLE III. DEDUCTIONS FROM AVERAGE OF CURVES FOR 5 AND 10-FT. FLOATS

Velocity of wind, mi. per hr.	5	10	15	20	25	30
Percentage which water travel was of wind movement	3.2	1.8	1.3	1.05	0.95	0.90
Wind movement in miles required to cause the water to travel 10 miles	313	555	769	952	1053	1111
Number of hours of wind at the given velocity required to cause the water to travel 10 miles	62.6	55.5	51.2	47.6	42.1	37.0

Lastly, from Table I the average of all the floats was used in plotting a curve, and from it Table IV was deduced.

TABLE IV. DEDUCTIONS FROM CURVES BASED ON AVERAGES FROM ALL FLOATS

Velocity of wind, mi. per hr.	5	10	15	20	25	30
Percentage which water travel was of wind movement	2.8	1.5	1.25	1.15	1.05	1.00
Wind movement in miles required to cause the water to travel 10 miles	357	666	800	869	952	1000
Number of hours of wind at the given velocity required to cause the water to travel 10 miles	71.4	66.6	53.3	43.5	38.0	33.3

Similar data were tabulated, using the north and south components of the wind movement and water movement, and the results obtained from plotting this second series differ but slightly from those given above. At a wind velocity of five miles per hour the percentage of water travel is practically the same, while for the higher velocities it shows an increased value.

Another plotting was made with the percentages which the water movement is of wind movement, as abscissas, and depths as ordinates. This showed that with the wind blowing at seven miles or less an hour there is a marked difference between the percentage of movement of the surface water and the deeper layers, while at the higher velocities no such marked differences exist.

Summary—A summary of some of the results of these experiments bring out the following:

The anemometer records proved conclusively that the south and southerly winds (the ones bringing pollution toward our supply) were the ones that blew for continuous periods the longest and the strongest, and also that the summation of the movement of the wind was generally always to the north by a very large amount.

The water movement induced in the lake by winds diagonal to the long axis of the same had a tendency to take a direction parallel to the axis; thus giving not only the south wind, but the wind from either south quarter, an opportunity of keeping the water in the surface of the lake moving toward our supply.

At the lower wind velocities, the floats prove that the percentage which water movement is of wind movement is not only greater than it is at higher velocities, but that the surface movement of water is greater than that of a lower stratum? So that if pollution were being carried in the upper stratum of water (as this is the place it must be received from the inflowing streams) the higher velocity of the upper layer of water will have a tendency to hold the pollution in suspension, and hence, carry it with it.

From the above review of the facts as noted the conclusion reached in this particular case, by very conservative interpretations of the experimental results, is that at many times polluted water could be brought from one end of the lake to the other, a distance of ten miles, in a period of from two to three days.

The Purchase of the Plant of the Chicago Reduction Co. has been authorized by the city council and mayor of Chicago. The price is to be determined by appraisal and the plant is to be operated under the direction of the health department. It is stated that the company asks $350,000 for the plant and $150,000 for the land on which it is located; also that the property of the company is assessed at a total of $126,541—$88,541 for real estate, $18,000 for personal property, and $20,000 for stock. The purchase of two sites for incinerative plants has also been authorized.

FIELD AND OFFICE

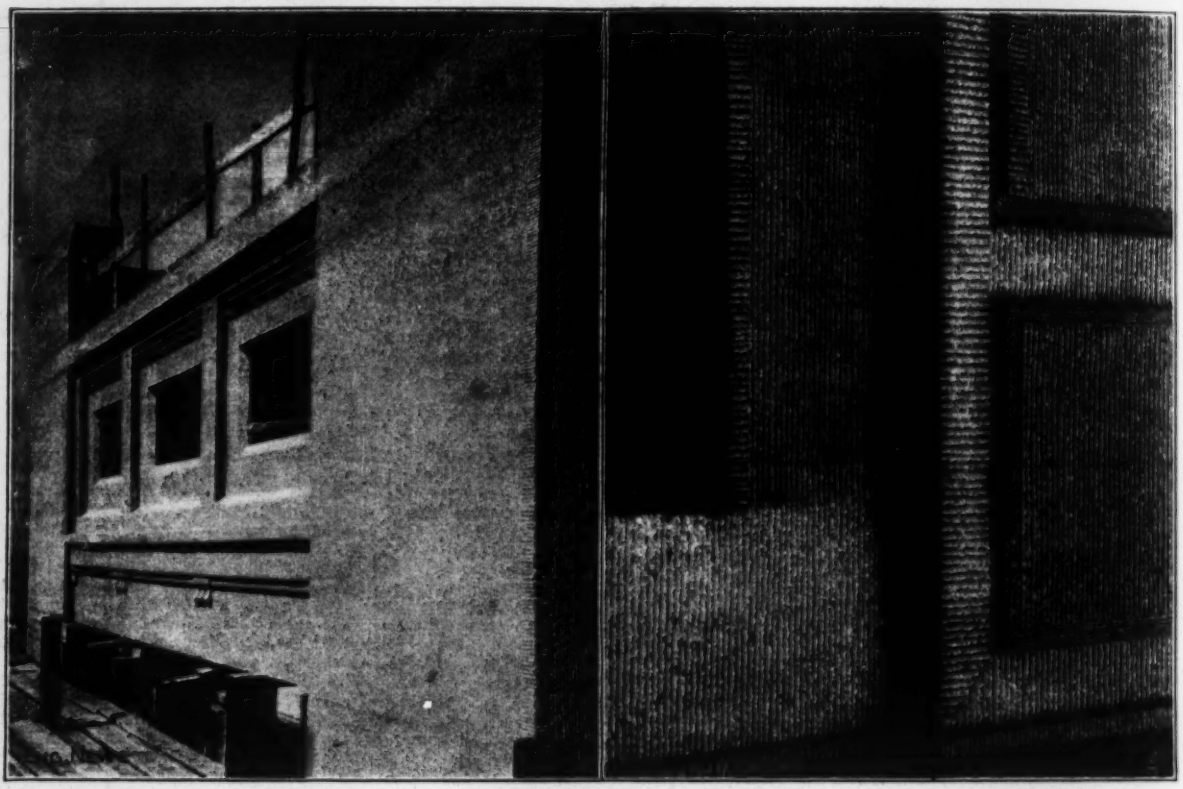
A Cut Surface-Finish for Concrete

The "Deutsche Museum," in Munich, Germany, the great technologic collection started five years ago, is having a magnificent reinforced-concrete building erected for it on an island in the Isar at Munich. The outside surfaces of the building are treated by chisel cutting in a remarkably effective way. The museum was organized as a result of the energetic work of Dr. Oskar von Miller (now president of the Verein deutscher Ingenieure), and hitherto its rapidly growing collections of historic technological exhibits, models, etc., have been housed in two old barracks buildings, one on Maximilianstrasse, the other on Zweibrückenstrasse. On an island just opposite the latter is the site of the new building, whose first half is now well along toward completion.

The new museum building was designed by the architectural staff of the museum. Mr. O. Gelius is architect in charge of construction. The structure is wholly of reinforced concrete, with girder, beam and slab-floor construction; it is founded on reinforced-concrete piles, part of them driven, part molded in place. The soil is sand; a loading of 50 tons per pile was used for the design. With the exception of a few stone mullions and columns, the whole exterior of the building shows the concrete face. To make this face uniform in texture and color, a novel chisel-cut finish is being applied; the results attained are highly excellent.

Cut Finish on Untreated Concrete Face

Figs. 1 and 2 exhibit the finish applied to the main part of the exterior. It consists of hand-cut parallel grooves, spaced about ½ in. At quoins the chisel cuts are made transverse (horizontal), as on drafted arrises in stonecutting; elsewhere they are vertical. This treatment gives a surface of even tone; the streaking and unevenness of the concrete as left by the forms are removed, and only large pockets or porous spots show through the chiseling.

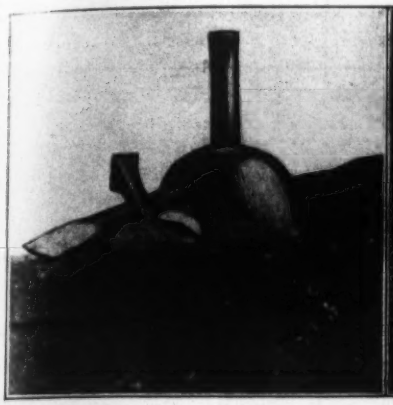


Figs. 1 and 2. Chisel-Cut Finish of Concrete Exterior, Deutsche Museum, Munich, Germany
(Straight-cut style applied to the concrete direct)

The cutting is done with a plain straight-bladed chisel 4 in. (10 cm.) broad. The worker holds the chisel at an angle of some 60° to the face, and strikes several blows, enough to make a triangular cut about ½ in. broad. He then moves it along 4 in. in the direction of the blade, and so prolongs the cut. The cut is made as long as the worker's position permits, and then the next adjoining cut is made. Figs. 3, 4 and 5 show, respectively, the chisel and mallet used, the second cut of a series started,

and a group of four successive cuts made. The original face texture of the concrete can be judged from Figs. 4 and 5.

This chisel-cut finish was worked out by Mr. Gelius through experimenting. He trained one man to the work, and this man then instructed others and has since been the piecework subcontractor.



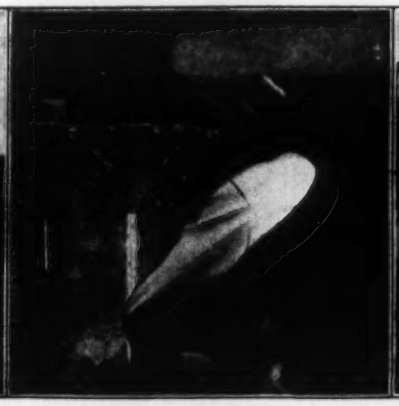


FIG. 3. TOOLS USED IN MAKING CUT FIG. 4. MAKING FIRST CUT FIG. 5. FOUR PARALLEL CUTS MADE
FIGS. 3-5. TOOLS AND METHOD OF MAKING THE CHISEL-CUT FINISH

A skilled man, it is stated, will cut 1 sq.m. (10 sq.ft.) in 1½ hr. The price paid to the general contractor by the Deutsche Museum is M1.40 per sq.m., or about 3½c. per sq.ft. The men, working on a piecework basis, get somewhat less than this amount.

The specification for the finish requires that the concrete must be at least 3 mo. old, but much of the work has been done on concrete 1 to 2 yr. old.

CUT FINISH ON TROWELED FACE

A variation of the straight-grooved cutting is used on some panel surfaces. It consists of patch-cutting as illustrated in Figs. 6 and 7. It is always done on a troweled plaster-coat applied on a very rough surface of concrete, with the result that the cutting exposes a large number of stone facets.

In Fig. 6 the successive stages of this finish may be seen. The original concrete face shows around two squares of the plaster-coat, which has some of the patch-cutting done on it. The concrete of the wall is placed with its face portion very poor in sand; the aggregate is crushed gravel, made from gravel dredged out of the Isar River, near the site, and containing many siliceous pebbles. This gives a face of very porous or open character, showing the stones closely set, projecting from a recessed matrix. The plaster-coat, which may be applied at any time after the forms are removed, is a 1:3 mix made with stone screenings. It is troweled with a metal float.

The cutting is done on this plaster-coat after not less than two weeks, and when the wall itself is at least two months old. It is done by making parallel chisel-cuts in squares about 8 to 9 in. on a side, with different alignment in the different squares. The chisel cuts through the plaster-coat and lays bare clean faces of the pebbles in the wall surface; but the interspaces between the stones show the mortar of the plaster-coat, and a ridge of this mortar also shows between the separate cuts. The general effect is well shown in Fig. 7, which also gives an idea of the manipulation of the chisel.

FIGS. 6-7. PATCH-CUT FINISH ON TROWELED PLASTER-COAT FACE, DEUTSCHE MUSEUM

This type of finish costs the same for cutting as the

straight-grooved style, but the cost of the plaster-coat is extra. The latter amounts to 70-80 pf. per sq.m., or about 2c. per sq.ft., the pay for labor (piecework) being 40 pf. per sq.m.

In applying these two styles of finish to curved and molded work no special difficulty was met, except that it was necessary for the men to train up on straight work.

A series of columns near the top of the tower was not cut in place, however. The columns were cast in a shop (not at the site), and the face cutting was done before shipping them; they were made of stone concrete, as against the broken gravel used for the wall work generally, and show rather finer grooving and somewhat different color from the wall work.

The balusters of the hand railing around the tower top were also cast in the shop. They were cut at the site, but before being set in place; the cap rail was cast over them after setting. The chisel work on the balusters took about twice as long as that on a similar area of flat work, and payment was at a special rate.

The railing just mentioned also includes some bushhammered face-work, on the solid corner panels. A fine, even texture was secured; the cut is rather deeper than on most bushhammered-concrete work in the United States. The cost is stated to be 15-20% cheaper than that of chisel-cut finish.

Labor Cost

In judging of the above given costs, it is necessary to take account of the large difference in labor cost between here and Germany. The usual wage-rates of three classes of mason labor at Munich are given as follows:

Stone mason (stonecutter)	M8½ per day ($2.10)
Bricklayer	M7 per day ($1.75)
Laborer	M5½ per day ($1.35)

The information given in this article was furnished by Architect Gelius who, as noted above, is responsible for the method and for its successful application to this particular building.

*

Short Cut to the Least Radius of Gyration

A convenient rule, easily memorized, for determining mentally the least radius of gyration of angles, without the use of a steel handbook, is the following:

$$r \text{ (minimum)} = cx(\textit{sum of legs})$$

For angles with equal legs, $c = 0.098$.

For angles with unequal legs,

½ in. difference in legs, $c = 0.096$
1 in. difference in legs, $c = 0.092$
1½ in. difference in legs, $c = 0.088$
2 in. difference in legs, $c = 0.084$
2½ in. difference in legs, $c = 0.080$

Except for 6x4-in. angles, $c = 0.087$; 8x6-in. angles, $c = 0.092$.

This rule gives results with a maximum error of 3%, irrespective of the thickness of the angle.

Examples:

4x4-in. angle, $r(\text{min.}) = 8 \times 0.098 = 0.78$
4x3-in. angle, $r(\text{min.}) = 7 \times 0.092 = 0.64$

This very simple method was developed by H. W. Aplington, 202 Riverside Drive, New York City.

Manual Signals for Field Engineer

A system of hand signals used by field engineers, t... sit parties, etc., on the Chicago, Burlington & Qu... R.R., was described in our issue of May 22, and ... have since received from Robert S. Beard* a copy ... another set of such signals. The special feature of ... former set was that only one hand was required ...

Manual Signals for Engineers

each indication, but Mr. Beard suggests that the numbers 2 and 11, 5 and 6, and 4 and 9 could be confused very easily. He considers that any advantage in the one-hand signal is more than offset by the increased distinctiveness of the two-hand signals in his own scheme, which is illustrated herewith. The right arm is used for numbers 1 to 3, the left arm for numbers 4 to 6, and both arms for numbers 7 to 0. In addition to the numbers, there are signals to indicate "cut," "fill" and "decimal point," and a group of signals for general instructions as follows:

*Fifth Floor, Waterworks Bldg., Kansas City, Mo.

Transit and Level-Party Signals

A. Hold Up or Give Line.—Wave a small horizontal circle high overhead with hand or rod.

B. Move to Right or Left.—Move hand at shoulder height in direction required; slightly for a small move, arm length for a big move.

C. Plumb Rod to Right or Left.—Extend arm vertically and swing slowly in direction required.

D. Set a Hub or Give a Turning Point.—Hold any visible object horizontally overhead with both hands.

E. Set on This.—With point of rod on tack, swing top of rod in small circle.

F. All Right or Go Ahead.—A half-circle swing of one or both arms overhead.

G. Come to Point Signaled from.—A full-circle swing with either arm.

H. Go Back.—Face to the right of man signaled to, wave both arms up and down, raising one while dropping the other.

I. Can't Get You.—Describe an "L" or inverted "T" by a vertical motion followed by a horizontal motion of the arm.

J. Move Target Up or Down.—Move hand up or down in direction required. Motion of hand means move 0.01; half-arm, 0.1; full arm, 1.0 ft. (For level party only.)

Note.—When calling numbers use "punky" for eleven.

Ditching Car; Baltimore & Ohio R.R.

The clearing of ditches and trimming of slopes in cuts is a class of work which forms an important item in railway maintenance-of-way, and the removal of the excavated material is one of the problems where traffic is heavy. The use of wheelbarrows does not permit of rapid progress, and it is only in special cases that it is advisable to employ a work train. For work of this kind, both in cuts and in tunnels, the Baltimore & Ohio R.R. is

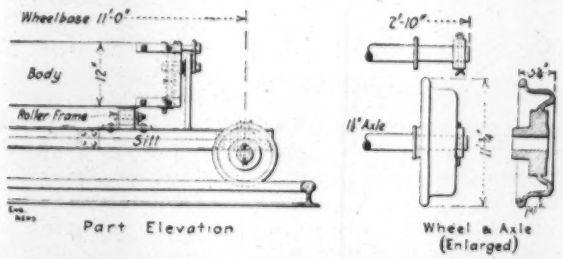


Fig. 1. Ditching Car for Track and Tunnel Work; Baltimore & Ohio R.R.

using a special type of dump car of its own design. This is shown by the accompanying cuts, and for information regarding it we are indebted to Earl Stimson, Engineer of Maintenance-of-Way.

A track of 27-in. gage is used, which can be laid between the ballast and the toe of the cut. The car (Fig. 1) has the body set low, so that it is easily loaded by the shovelers. It is dumped by sliding the body laterally on rollers and then tilting it. The side or door rises clear of the body as the latter tilts, so that the contents slide out beneath the door, as in many makes of large side-dump cars. The car has two sills 12 ft. long, of I-beam section (built up of angles and a web plate), connected by transoms. Pairs of U-bolts through the sills carry the bearings for a pair of axles with wheels 9¾ in. diameter on the tread and 11¾ in. over the flanges. The wheelbase is 11 ft. Across the sills are four frames each carrying three rollers 3 in. diameter and 3 in. long, 9⅞ in. apart. The body is of wood 9x3 ft., and 1 ft. deep.

Across each end of the frame is a bar or guide, having its middle portion horizontal and the ends curved down to their attachment to the sills, as shown in Fig. 2. Against the underside of this bar rides a roller on the end of the car body. When the body is pushed laterally across the frame, it remains horizontal for the first part of its travel, and is then tilted and dumped as the roller

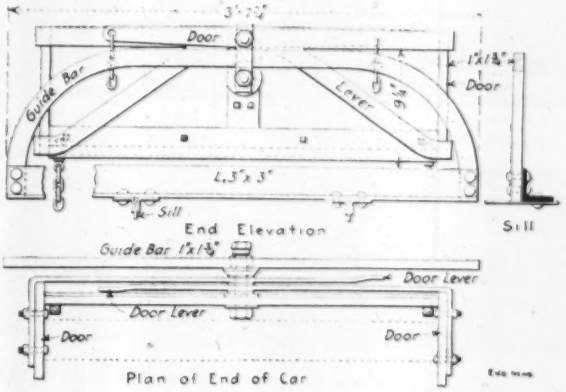


Fig. 2. Details of Ditching Car; Baltimore & Ohio R.R.

engages the curved end of the guide bar. Each side or door is carried by a pivoted Y-shaped arm, the fork being attached to the door and the stem being attached to the sill by a chain. As the body tilts the door swings upward. The weight of the car is about 800 lb., and three men push the car to the dump. One car will suffice for a gang of five men, and the length of haul is usually from 500 to 1500 ft.

This car is used extensively over the Baltimore & Ohio R.R., and has been found a very economical device. Each year a program is prepared for ditching work, and the ditching of cuts and widening banks is carried on continually as a part of the general improvement work.

Repairing the Trainshed Roof of the Union Station, at Chicago, became necessary some months ago on account of corrosion of members by smoke and gases, and also on account of some displacement of the trusses. The trainshed is about 31 years old, and was built of iron instead of steel, which is believed to account for its long resistance to corrosive action. The carrying out of the repair work was done under difficult conditions, due to the lack of room, the continual passage of trains, the movements of crowds of passengers on the platforms, and the suffocating smoke and gas from the engines. This last was particularly serious owing to the roof being low, so that the men on the working platforms were close to the engine stacks. At frequent intervals men would have to drop their tools and get away from the blast from a locomotive.

The general scheme of the work was to erect heavy bents of timber posts, carrying lines of I-beams on which a working platform was erected. This platform extended across the entire width of the shed, being moved forward as the work progressed. All material and new members had to be hoisted by machinery.

The repairs were planned and executed under the direction of the engineers of the Pennsylvania Co., which owns the station. The work was done by contract on a force-account basis.

Hydrographic Surveying; Oakland Harbor Development, California

By Fred W. Johnson

In connection with the extensive development of the harbor of Oakland, Calif., by the city authorities the following method of hydrographic surveying has been used. This method has proved the best and most economical under existing conditions.

The purpose of the surveys was to control and to check the dredging of a channel 9000 ft. long from the navigable waters of San Francisco Bay to three newly constructed timber-apron wharves along the rock-faced earth levee recently built between the mole of the Southern Pacific Co. and the trestle of the Key Route System (see Fig. 1). The depth of the channel is 25 ft. at mean low tide, with a bottom width of 350 ft. and side slopes ranging from 2 on 1 to 4 on 1. This channel is nearly parallel to the Long Wharf of the Southern Pacific Co.

The first step was the establishment of an accurate reference line, which was surveyed between the tracks on the wharf, and from this line a base line was calculated and called the *S* line. The *S* line is parallel to the center-line of the channel, and the stations of the *S* line are placed at right angles to the center-line on the stringers of Long Wharf nearest to the channel. The station numbers were 6 in. in height, stenciled in black on white boards nailed to the stringers to clearly mark out the lines of the ranges to be sounded.

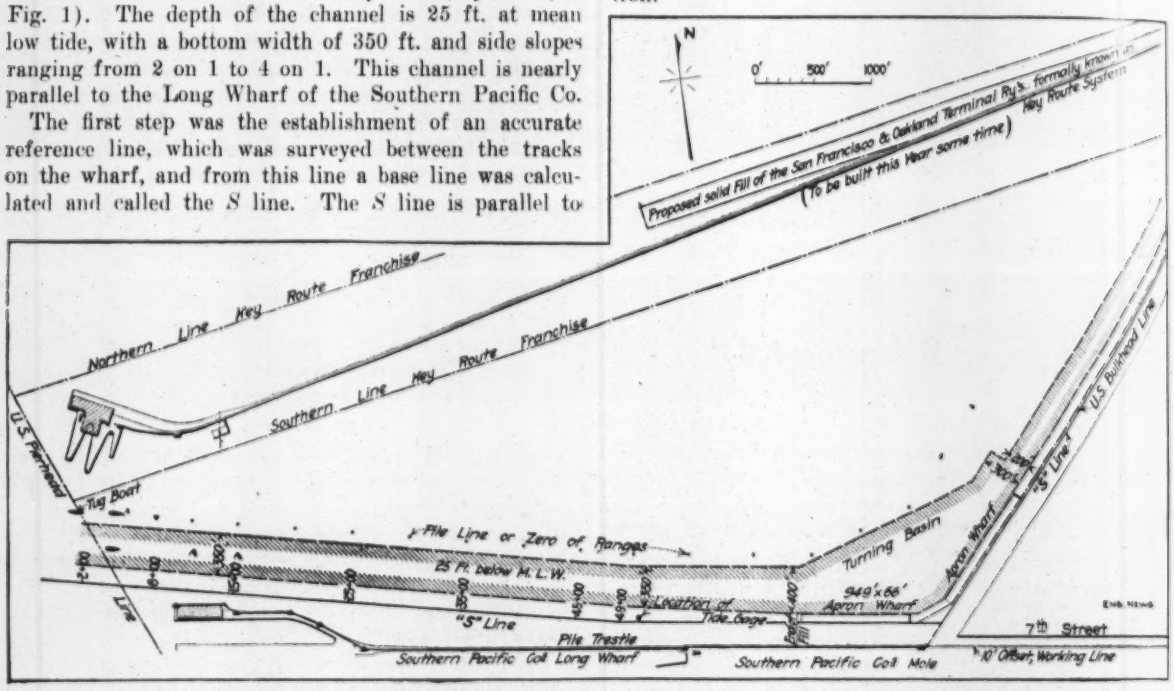


Fig. 1. Map of Oakland Harbor Showing New Channel and Survey Lines

From the wharf reference-line piles were located by triangulation 100 ft. north and parallel to the north limits of the channel. These piles were driven opposite to station marks on the *S* line at distances apart varying from 300 to 600 ft. and projecting about 5 ft. above high tide.

Owing to unsatisfactory records obtained from the readings of a staff tide-gage a self-registering or automatic tide-gage, such as has been in use for many years by the U. S. Coast & Geodetic Survey, Fig. 2,† was located about the middle of the work, so that at no time were the soundings taken farther than 5000 ft. from the tide gage. A staff gage was also provided to check the automatic gage readings from time to time. Still further to check the staff and automatic gages a float self-reading gage was installed.

The hexagonal float well was 16 in. between faces and extended from the tide-gage box to 10 ft. below the lowest low waters, so as to break the wind and surface-wave actions on the float. At first only one bottom was used, but after experimenting a second bottom was placed 4 in. from the first to doubly break the flow. The outer bottom with 5 holes 1 in. in diameter were calculated to more than equal the change of flow, while the inner bottom having 5 holes was calculated to take care of the flow plus 10%. Both bottoms were of ⅛-in. galvanized iron.

The automatic gage was housed in a large waterproof wooden box with hinged top and side to allow free access to the gage for adjustments. The box was built on two piles with the float well between them. Around these piles were four more piles, making a 10-ft. square guard to protect the gage from being jarred or damaged by driftwood or small craft striking the piles carrying the gages. Both automatic and float gages have been in use constantly for over a year and no serious re-adjustment has been necessary. Seaweed has had to be removed from the bottoms of the wells about every four months.

A working datum plane was arbitrarily established 9.8 ft. below the true datum plane or city base, which is 6.8 ft. above mean low tide. To correct minus readings caused by extreme low tide all the gages were set to the new datum plane. For convenience of scaling and adjusting the reference plane of the automatic tide gage, the tracer was set at +1 of the working datum plane.

The hydrographic party consisted of a recorder, leadsman and three boatmen. The sounding outfit consisted of two 15-ft. boats, one used by the sounding party and

*Office Engineer, Harbor Dept., City Engineer's Office, Oakland, Calif.

†The report containing a description and principles of the automatic tide gage may be obtained by applying to the Superintendent of the Coast & Geodetic Survey, Washington, D. C.

the other having a reel bolted to its stern was used to carry and erect the working or layout lines.

The two lead-lines, 40 ft. long, were made of No. 8 brass-jack chain fastened to an 8½-lb. iron weight having a bottom 4 in. in diameter to keep it from sinking in the soft mud. The lines were marked each foot with cloth markers, the tenths being judged by the eye.

Two sets of blocks and tackles with clamps to string the range and pile lines (to be hereafter described), a sextant, a right-angle instrument, wire pile line and two wire range lines completed the equipment.

The wire pile line, 600 ft. long, was stretched between two piles marking the north offset line of the channel. This wire had a 3-in. hook at one end and was marked off into 50-ft. spaces. The pile line was stretched between the two piles by hooking the end into a rope sling around one of the piles. The reel boat was made fast to the second pile and by means of the stern reel as much slack in the wire as possible was taken up. Then a block and tackle fastened to a rope sling on the pile were used to string the wire tight by grasping it at a convenient point with an ordinary lineman's clamp, known as a

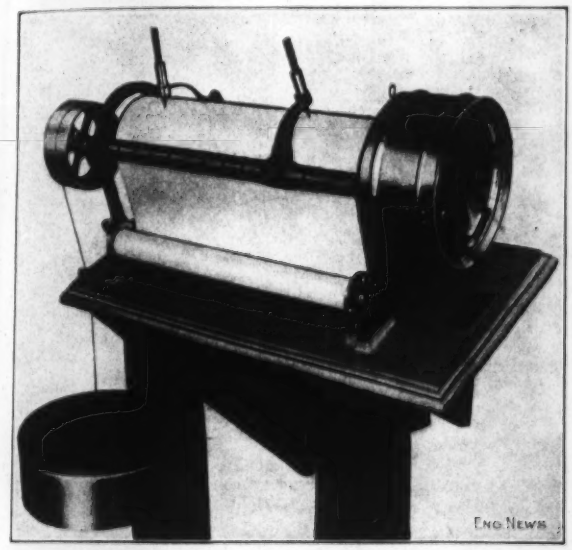


Fig. 2. Recording Tide Gage Used in Oakland Harbor Surveys

"come-along." The rope slings on the piles were placed about 5 ft. above the surface of the water, allowance being made for the rise and fall of the tide, as the lines were taken down each day.

The wire range lines, 900 ft. long, were stretched between the pile line and Long Wharf. This wire was equipped with 3-in. hooks at one end and marked off into 25-ft. spaces.

The markers of all lines were fast colors of red, black or white cloth, used separately or in combinations, tied to copper loops fastened into one of the strands of the wire-rope line. Several makes of wire rope were used, but the most satisfactory was a ⅛-in. 19-strand galvanized-wire aviator cord, weighing 3.6 lb. per 100 ft. with a breaking strength of 2300 lb.

To the pile line the range lines were fastened by sister or matched hooks, which were held at the right stations by first winding a cloth around the pile line to prevent cutting and then wiring the matched hooks on it with coils of copper wire.

The loose end of the range line was taken over to Long Wharf and fastened to the correct center-line station by making the reel boat fast to a wharf pile and taking up on the reel as much slack as possible. The block and tackle were then fastened to the wharf by wire slings and the slack taken up, which in the case of the range lines was about 30 ft.

All the lines were marked by use of a steel tape after the above pull had been applied, which made the distances nearly constant the entire lengths. The first soundings were taken on the true pile line which was called the zero of the range. Due to the sagging of both the pile and range lines each range had an equation to carry along. This equation was always measured by tying a tape to the pile line and measuring the distance back to the true pile line. The boat being 15 ft. long made it easy to apply this equation by either adding or subtracting from each 25 ft. of the range line the proper distance to make the bow of the boat, where the sounding lead was dropped, come to the correct distance from the true pile line or zero of the range.

After the boats with the wire-rope lines were taken out to the section to be sounded, it took an average of 30 min. to put the lines up. Each range was sounded in an average of 15 min., beginning at the pile line and ending at Long Wharf. It required from 12 to 19 min. to change the range line to another station.

In changing the range line the block and tackle was loosened, and the end of the range line on the pile line was moved along, after loosening the matched hooks, to the next station and made fast again by the sounding-boat party. The reel boat was then moved to the next position on the wharf corresponding to the next station on the center line, and the line was drawn up as described above.

In sounding along the range line the wire rope was held out of the water by running it through an oarlock, and unless the current was very strong, the boatman moved the boat along by pulling on the range line, otherwise he rowed enough to overcome the force of the current. The current ran about 4 miles an hour and at 45° to the course of the channel.

Soundings were never taken after the water became choppy, and every afternoon about 2:30 o'clock a southwesterly wind made it too rough to continue work that day.

The bottom before dredging, was sounded in rectangles of 25x50 ft.; the 25-ft. sides were along the ranges and the 50-ft. sides were along the *S* line. In the final soundings the limits of the slopes were located as nearly as possible, and were then sounded at 5-ft. intervals until the bottom of the channel was located, then 25-ft. distances were used across the channel. The time of each sounding was recorded to the nearest minute in all the work.

From Sta. 2+00 where the 25-ft. contour began to Sta. 7+50, owing to the course taken by the ferries (every 20 min.) of the Key Route System, and the landing of ships at the north arm of Long Wharf, piles could not be driven, so the method described was changed somewhat.

The soundings were taken parallel to the channel instead of at right-angles to it. Range lines were stretched

from the second and third piles on the true pile line to Long Wharf, but beyond the pile line an extension was stretched from the first pile, which was a dolphin, to a point just outside of the United States pierhead line, and this line was held taut and on line by an anchored tug boat (see Fig. 1).

Large range flags were placed 25 ft. out from the pile line on each of the last two range lines marked by piles and at each of the successive pairs of 25-ft. distances until the entire distance across the channel had been sounded.

The sounding party rowed out on line with the range flags and when a flag held up by a man in the reel boat at each of the successive 50-ft. distances of the pile line was sighted with a right-angle instrument, the sounding was taken and the time recorded. This method was a little faster than the method first described, but hardly as accurate.

The final soundings were not taken until the first section of 5100 ft. had been dredged. Monthly estimates were made up from notes obtained at least once daily from soundings taken around the dredge. The position of the dredge was located with a sextant by noting the angles between any two piles and a station on the wharf, or two stations and a pile, taken from a certain position on the deck near the spud. Soundings were taken around the dredge at the extreme of the swings giving two sets of soundings for the same sextant tie. To be sure that the bottom was to grade, random soundings were taken at the first of each month.

NOTES

Quick Lime for Thawing Frozen Ground—In describing some difficult sewer construction at West Liberty, Iowa, at the annual meeting of the Iowa Engineering Society last February, Charles P. Chase, the engineer in charge, gave the following details regarding the use of lime to overcome frost:

The work was carried on through the whole winter, when the ground was frozen to a depth of 4 ft., in which state it resisted all efforts of a trenching machine to break through it. The lime was spread over the frozen ground, covering the width of the trench to be opened, and was then broken up into small pieces and covered thickly with straw, hay or manure. Water was then poured on the straw so as to thoroughly slack the lime; hot water was found best and hastened the slacking. The covering retained the heat, which, with the hot water, penetrated the frozen ground sufficiently to enable the trenching machine to make headway. On another job a covering of old boards with a steam jet was used to hurry up the process. The same process has been successfully employed in Clinton, Iowa, where the contractor continued work without interruption throughout the winter.

An Automatic Blueprint Finisher—Concerns supporting large drafting rooms, and those whose business it is solely to make blueprints, should be interested in a machine recently brought out which washes, dries and irons blueprints. Immediately after exposure to the light the print is dropped on a bed of continuous belts which carry it under a spray of fresh water. Following this, the spraying operation is repeated and the print is then passed on by means of other belts to a drying and ironing drum.

The used water flows back beneath the belt to a storage tank from which it is pumped up for use again. After the second spraying on the wet belts the print passes onto a series of dry belts to receive a preliminary or partial drying from a stationary heater (usually gas). From there it is carried on a heated metal drum and the drying and ironing process is completed. After almost a complete revolution the print is released from the drum and is delivered automatically at the back of the machine onto a table ready for use.

It is claimed that a 15x20-in. print on the lowest of the three speeds will require for the complete operation a little over 2 min. and on the highest speed a little over 1 min. The 42-in. machine occupies only 45 sq.ft. of floor space, the height over all being 7 ft. 6 in. It is driven by a 1-hp. electric motor. The cost of operation for a 10-hr. day is claimed to be about $3.20, this is allowing $1.50 a day for the operator, at 75c. a thousand feet and electric power at 8c. per kw.

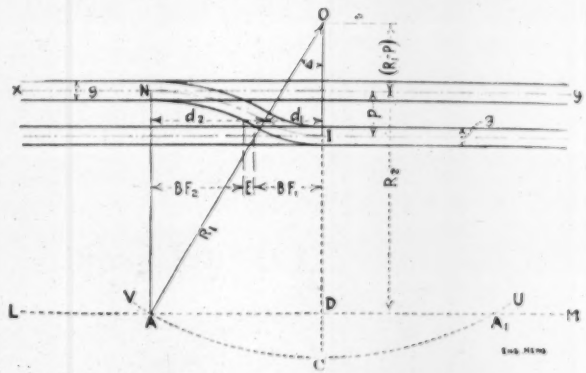
The apparatus is called the Levi Automatic Blueprint Finisher and is manufactured by the Technical Supply Co., Scranton, Penn.

Graphical Solution of Crossover Problem—Part of the accompanying figure appeared in "Engineering News" of June 24, p. 172, giving the solution of a crossover problem by H. Knouse. The solution for the computation of Δ of this problem is given in Nagle's "Field Manual for Engineers," p. 1[illegible] formula (116).

A graphical solution is offered here, which can be used in lieu of computations or as a check. Take I as the P.C. of crossover to left. Erect perpendicular IO = R_1. With O as a center and a radius = $(R_1 + R_2)$ describe arc VU.

Draw LM parallel to the center line of the tracks, and at a distance R_2 from XY. Then A, the intersection of these two loci, is the center of the crossover to right having a radius R_2.

Where OA crosses the arc of R_1 determines the P. R. C.



GRAPHICAL SOLUTION OF CROSSOVER PROBLEM

and angle Δ can be measured with a protractor. Erecting a perpendicular AN to XY fixes the P. T. By describing arcs with radii $(R_1 + \frac{1}{2} g)$ and $(R_2 + \frac{1}{2} g)$ where g is the gage and noting intersections, BF_1, BF_2 and E can be measured.

Inspection of the figure shows DC = p, consequently we can write at once, vers $\Delta = \frac{p}{R_1 + R_2}$. Also $(d_2 + d_1) = AI = \sin \Delta\ (R_1 + R_2)$; and $E = \sin \Delta\ (R_1 + R_2) - (BF_1 + BF_2)$.

Concrete Water Barrels for Railway Bridges are described in a paper by Hunter McDonald, Chief Engineer of the Nashville, Chattanooga & St. Louis R.R., in the 1912 "Proceedings" of the Engineering Society of the South. One design is 30 in. deep, 24 in. outside diameter at the top, tapering to 22 in. at the bottom. The sides and bottom are 2 in. thick, reinforced with a sheet of expanded metal. The cover is of iron, hinged to a lug embedded in the concrete, and is depressed so as to catch rainwater. They do not leak, the evaporation is slight, and the concrete will resist bullets (which are apt to be fired at the barrels by boys and others). The total cost is about $1.70, divided as follows:

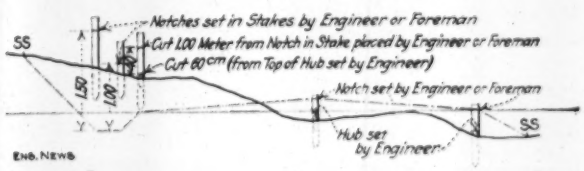
Cover: Galvanized iron, 5 sq.ft. at 3c	$0.15	
Hinge, bolts, rivets and wire	0.05	
Cost of making, 1 man 1 hour	0.20	$0.40
Barrel: 17 sq.ft. exp. metal at 3c	0.51	
0.10 cu.yd. 12:4 concrete	0.21	
Forms (25 barrels to one set)	0.10	
Labor	0.30	1.13
Engineering and contingencies, 10%		0.13
Total		$1.70

Other experiments have been made, using a less tapered form of barrel, galvanized-iron forms, and poultry-netting reinforcement. Plastering the mortar on the reinforcement has not resulted in tight barrels, but 1:2:4 concrete cast and rammed into forms has given good results. It is believed that good concrete water barrels, with metal tops, can be made for $1.30 each when made in large quantities. Another design consists of a concrete box let into the ground all but about 1 ft., this upper part being marked with the number of the trestle.

The wooden barrel is the more expensive of the two, considering its capitalized cost on the basis of six years' life, leaving out all consideration of increased cost of maintenance. The ballasted-deck trestle is rapidly becoming the standard on most roads, and it is thought that in the future the water barrel will seldom be needed. For this reason the concrete barrels are not used to any extent on the N., C. & St. L. R.R.

Task System of Constructing Roads, Philippine Islands—The task system, or "paquiao system" as it is called in the Philippines, consists of letting out small pieces of work to squads of native laborers at a stipulated price; the work to be done in a given contract time. The same method has been practiced to considerable extent in this country in railway construction under the name of "station" work. The task system proved so successful on the construction of the Dueñas-Lambunao Viejo Road that most all the road grading in Iloilo Province is now done by this method, and only enough day laborers are retained to do stations that are too small to "paquiao," and to finish such stations as are not completed by the haul from the cuts being done by contract. This work is described in the April issue of the "Quarterly Bulletin" of the Philippine Bureau of Public Works; and a number points there brought out in regard to handling ignorant laborers and making their work clear to them will prove of value where similar methods are employed.

There are several details in connection with this class of contract work which are more important than in the day-labor system. In the first place, the foreman must be reliable and painstaking, and must understand handling local labor. Hubs must be driven at each break in the cross-section of the roadbed, as in the accompanying illustration, and all cuts and fills marked with reference to the top of these hubs. It is not sufficient to take the elevations of the top of the ground and mark the cut or fill, because the stations are scarcely ever finished consecutively and discrepancies which



would arise because of the unevenness of the ground would produce irregularities in the roadbed, which in case of day labor are avoided by stretching grade lines. When these hubs are set the foreman or his assistants can set stakes and make notches in them at even meters or fractions thereof above or below the finished points of the grade, and the "taos" will soon learn to do the rest.

Aside from the setting of the hubs and the figuring of the quantity of each station, with the information as to haul, the success of this kind of work depends entirely upon the foreman. The foreman on the Dueñas-Lambunao Viejo Road has been extremely successful in this work. The following notes gathered from the foreman's point of view will give a good idea of the most successful manner of handling "paquiao" work:

"Paquiao" with the laborers direct, if possible, and not through a middleman.

Occasionally a gang will ask that the price of their work be increased, claiming that they are making less than ordinary wages, but generally, if looked into, it will develop that they have made a mistake in figuring their shares.

As a rule, they will want to turn in their work a little short of completion, and a foreman must be well acquainted with the job, for where a large number are working they will skimp the work unless apprehended. A little jollying goes a long way with the laborers; but one needs to know and understand them to do this effectively. They are ever ready to be "shown."

In "paquiao" cuts and fills it is always better to contract the cuts, arranging in the agreement for the dirt to be so deposited that the stations to which the dirt is hauled will be completed—that is, not allowing more than one station of fill to be incomplete when the cut is finished; then if there are any stations left for fill, they can be "paquiao" as "borrow and fill," and the remaining station can be finished by day labor. In this way many stations of fill are often completed free of charge where the haul is not more than 40 m.

Safety Rules for Handling Construction Materials—The following rules are contained in a general article on "Safety, Relief, Sanitation and Welfare in the United States Steel Corporation," published in the June issue of the "Journal of the Engineers' Society of Pennsylvania":

RULES FOR PILING BRICK—(1) Except in brick sheds, brick shall not be piled higher than 7 ft.

(2) The pile shall be tied at every course with alternate courses of headers and stretchers.

(3) When the pile is over 4 ft. high, it shall taper back from a point 4 ft. high, 1 in. to each foot.

(4) In unpiling, the taper shall be maintained.

(5) Brick shall never be piled directly on the ground; planks shall be used as foundations.

(6) Under no circumstances shall brick be piled for storage purposes on scaffolds or runways.

(7) Tie strips of wood shall be inserted wherever necessary.

(8) All foremen in charge of piling brick shall be held individually responsible for safety of the piles.

RULES FOR PILING CEMENT—(1) Cement shall not be piled more than ten bags high, except in storage built for such purpose.

(2) The first four end bags to be cross-tied in two separate tiers up to the fifth bag, where a step-back of one bag in every five bags shall be made. Beginning with the fifth bag only one cross-tier will be necessary.

(3) The back tier should, when not resting against a wall of sufficient strength to withstand the pressure, be stepped back one bag in every five bags, the same as end tiers.

(4) In storage, when piled between and against walls of sufficient strength to withstand the pressure, no cross-tiers nor step-backs will be necessary, but the bags should be piled with a slight incline against the back walls, the height depending on the strength of the wall.

(5) Cement bags in the outer tiers shall in all cases be piled with the mouth facing the center of the pile.

(6) When cement is removed from a pile, the length of the pile shall be kept at an even height, and necessary step-backs, every five bags, be taken care of.

Plumb Line Target—The accompanying illustration is self-explanatory. The target is made of white celluloid. It is of convenient size to carry in the pocket and is readily attached or detached to the plumb-bob line by the slots shown in the illustration. It is claimed to be most appreciated when sighting in dark or shady places, or toward dusk when the light is failing. In such cases a match held behind the cut in the target will enable the transitman to quickly locate the plumb line. It is also claimed to be particularly serviceable where the background is dark as in the case of grass or foliage. The only possible objection we can see to its use is the resistance it would offer in a breeze and hence make it more difficult to center the bob. This device is called the "Fulton" pocket target and is made and sold by Kolesch & Co., 138 Fulton St., New York City. The price is 35c. each or $3.50 per dozen.

The Use of Cement in Bulk was tried on a part of the track elevation work on the Bloomingdale Road line of the Chicago, Milwaukee & St. Paul Ry. (at Chicago), which was described in our issue of July 3. From information furnished by the Universal Portland Cement Co., it appears that when the cement was shipped in sacks, two men were required in the car to place the sacks on a platform, from which three men loaded them onto wheelbarrows and wheeled them to the mixer. There another man untied the sacks, emptied them into the charging chute and laid them aside to be counted, bundled and shipped. Five men (sometimes six) were required to keep the mixer supplied with cement. In this case the supply train was made up with the sand and gravel cars between the cement car and the mixer.

When handling bulk cement, the cement car was placed next to the mixer (which could be done also with sacked cement), and ordinarily two shovelers in the car and two wheelbarrow men could handle the required amount. This dispensed with one wheelbarrow man and one man on the platform, while the work of caring for the empty sacks was eliminated also. The above force was increased at times, however, by a third shoveler and sometimes a third wheeler also. This was especially the case in starting work for the day in order to get the mixer working rapidly, as the operation of the mixer governed the speed of the entire gang and was itself dependent upon the pace set by the men supplying the material to the mixer. Thus the number and distribution of the men in the gang varied from time to time.

There was comparatively little dust caused by handling the cement, either in the car or on the mixer platform. The wheelbarrows when filled in the car had the cement struck off level with the sides, giving a net load of 2 cu.ft.

Surveying Instruments From a British Point of View—In a recent discussion before the Institution of Mining and Metallurgy, London, England, D. J. Browne, of Rossland, British Columbia writes:

My personal experience has been mainly with mining "transits" of American manufacture. The most serious faults of American theodolites are:

(a) Four-screw leveling; (b) insufficient centering play; and (c) faulty design of standards.

(a) Although standard triangulation and geodetic instruments are provided with the three-screw leveling arrangement, the latter are only supplied to order for engineering theodolites; usually at an extra cost. The fault probably lies with the surveyors, many of whom, no doubt, overlook the fact that the extra rigidity may be attained at too great a cost.

(b) The "shifting center," usually associated with four-screw leveling, is the only centering device in use. Its range of movement varies from $\frac{1}{8}$ in. to $\frac{3}{4}$ in., according to the size of the instrument. The mechanical centering stage is very rarely seen.

(c) Owing to the presence of a large compass circle, the feet of the standards are crowded to the edge of the plate. This compass is, I believe, a relic of the dial used in the early days of railway development, and owes its survival to railway locating engineers, who are accustomed to check azimuth readings roughly by its means. On a mining theodolite, however, it is out of place; and the space it occupies should be devoted to giving lateral strength to the standards. Some of the more enterprising makers have recognized this, and are now introducing the well known U-form of standards, and are either dispensing with the compass entirely, or substituting the trough form mounted elsewhere on the instrument.

In other respects American instruments have much to recommend them. They are light and easily manipulated, and the circles are generally accurately divided and easily read. The azimuth circle is invariably protected from dirt, moisture and injury by an extension of the vernier plate with glazed verniers; and the altitude circle is generally provided with an aluminum guard. Apart from the defects indicated above, the American theodolite is a good surveying tool.

As to British instruments, L. H. Cooke, of the Royal School of Mines, South Kensington, London, writes:

At a distance of 50 years lies the origin of the first transit theodolite specially designed by a British mine-surveyor to suit mining conditions. Between 1863 and 1890 stretches a barren waste of years in which no new mine-theodolite of native origin has quickened into full life, no foreign form been translated and taken root.

True it is that attempts have been made by mine-surveyors and instrument-makers, independently or in collaboration, to devise new forms, but not one of them has survived the frosts of our national indifference and neglect of publicity. No British mine-surveyor has afforded public guidance to the instrument maker.

The British mining schools have been unable to find time and means properly to treat the subject of instrumental construction in their courses; they have been unable to originate improvements themselves; they have been forced to neglect the fostering of improvements of others by their investigations, criticisms, testimony and support.

A Sledge for Breaking Rock—In these days it is rare that hand crushing of rock has to be resorted to, but there are times when it will prove more economical, and other times where it is the only method available. For such occasions, a special sledge quite generally in use in mining practice has been called to our attention. This sledge may be common enough in some parts of the country, but we think it will be novel to most engineers who have not had mining practice. It can best be described in the words of E. D. Peters, Jr., in his book, "Modern Copper Smelting."

In breaking rock, there should be several cast-steel sledges, differing in weight from 6 to 14 lb., intended for general use in breaking up the larger fragments of rock to a size suitable for the light spalling hammers. Each laborer should be provided with one of the latter hammers 6 in. in length, forged from a 1¼-in. octagonal bar of the best steel and weighing about 2¾ lb. This should be somewhat flattened and expanded at the middle third to give ample room for a handle of sufficient size to prevent frequent breakage. The handles usually sold for this purposes are a constant source of annoyance and expense, being totally unsuited for this peculiar duty. It is better to have the handles made at the works, if it is possible to procure the proper variety of oak, ash, hickory, or best of all, a small tree known in New England as ironwood (hornbeam), which when peeled and used in its green state, excels most other woods in toughness and elasticity. The handles should be perfectly straight without crook or twist, so that when firmly fastened in the eye of the hammer by an iron wedge, the hammer will hang exactly true. Their value and durability depend much upon the skill with which the handles are shaved down to an area less than half their minimum size, beginning at a point about 6 in. above the hammer head and extending for about 10 in. toward the further extremity. If properly made and of good material, they may be made so small as to appear likely to break at the first blow, but in reality they are so elastic that they act as a spring and obviate all disagreeable effects of shock, wear longer and do more work than the ordinary handle.

We are informed that with such a hammer, with a handle perhaps three feet long, rock can be broken down to concrete aggregate size with great facility.

⁂

Formulas for Finding the Intersections of Wing Walls with the Main Abutments of Bridges

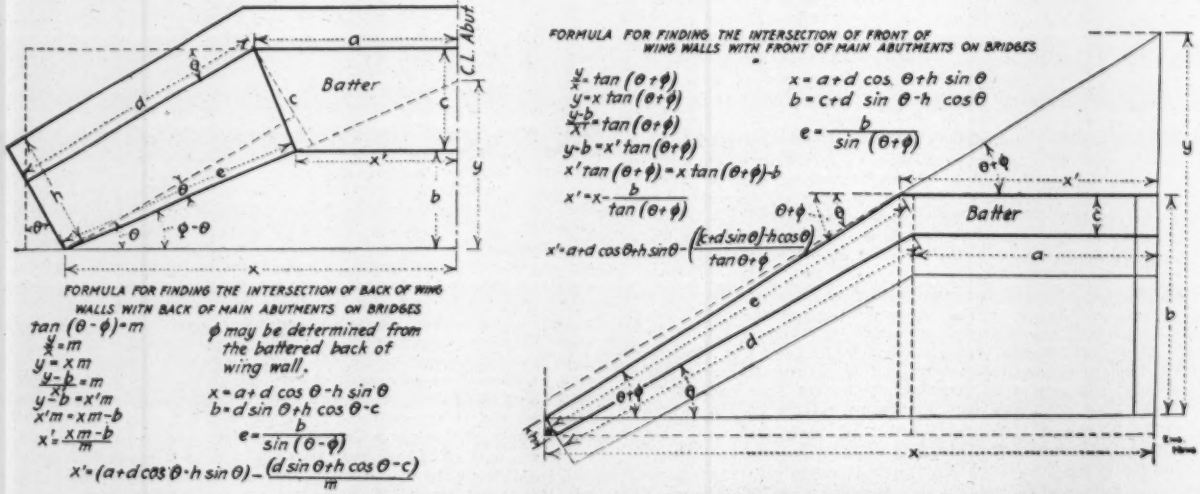
By S. Reswick*

I have frequently noticed that even good draftsmen and computers have some difficulty in finding the intersection of the batter lines of a bridge abutment with those of its wing walls†, when the latter make other than angles of 180° with the main body of the abutment. To facilitate these computations the accompanying formulas were developed, and they have proved to answer the purpose in our office quite successfully.

These formulas give the intersections in terms of quantities which are known or can readily be found. There are two formulas, Fig. 1 and Fig. 2, one for the back and the other for the front of the abutment, respectively. Their derivations are plainly indicated, and the interested reader can easily prove them and test their usefulness.

*New York State Barge Canal, Fulton, N. Y.

†Other recent articles on the design of abutments and retaining walls may be found in "Engineering News" of Nov. 21, 1912, p. 969, and Dec. 19, 1912, p. 1148.


C.L. Abut
Batter
FORMULA FOR FINDING THE INTERSECTION OF BACK OF WING WALLS WITH BACK OF MAIN ABUTMENTS ON BRIDGES
tan (θ-φ)=m
y/x=m
y=xm
(y-b)/x'=m
y-b=x'm
x'm=xm-b
x'=(xm-b)/m
φ may be determined from the battered back of wing wall.
x=a+d cos θ-h sin θ
b=d sin θ+h cos θ-c
e=b/sin(θ-φ)
x'=(a+d cos θ-h sin θ)-(d sin θ+h cos θ-c)/m
FORMULA FOR FINDING THE INTERSECTION OF FRONT OF WING WALLS WITH FRONT OF MAIN ABUTMENTS ON BRIDGES
y/x=tan(θ+φ)
y=x tan(θ+φ)
(y-b)/x'=tan(θ+φ)
y-b=x' tan(θ+φ)
x' tan(θ+φ)=x tan(θ+φ)-b
x'=x-b/tan(θ+φ)
x=a+d cos θ+h sin θ
b=c+d sin θ-h cos θ
e=b/sin(θ+φ)
x'=a+d cos θ+h sin θ-((c+d sin θ)-h cos θ)/tan θ+φ
Batter

FIGS. 1 AND 2. FORMULAS FOR WING-WALL INTERSECTIONS

EDITORIALS

Civil Engineer or Dentist?

The latest contribution to the vexed question of raising the status and emoluments of the engineer comes from Buffalo. It takes the form of advice to be a dentist rather than a civil engineer.

It came about this way. "Anxious Mother," living near Buffalo, was in a quandary because one friend had advised that her 16-year old son become a civil engineer while another was sure dentistry would be better. On appealing to the Editor of the Buffalo *Evening News* she was given this opinion in the issue of that Journal dated Aug. 14, 1913:

> Of the two professions you mention, we think that the supply of civil engineers is pretty nearly up to the demand. The dentists, who are well paid for their work, have all they can do. One dentist that we heard of charges $3 per hour for cleaning teeth, which is more than he could get as a civil engineer. The dentists also fix their own scale of prices and are not governed by anybody. They manage to get all they can. Circumstances, surroundings, location of offices, govern their prices. On the whole, dentistry is a very profitable profession. Your son should write to the Dean of the University of Buffalo, Dental Department, who could tell him more about it. With steady employment at $3 per hour, it would be in a day or 10 hours' work, $30, which cannot be gotten in any other profession. They are always busy or they pretend to be busy and it is necessary to make an appointment three or four days ahead with some of them to have any work done.

The more one ponders the foregoing advice the more of a Solomon does the Editor of the Buffalo *Evening News* seem to be.

⁂

Good Roads Extravagance and Good Sense in West Virginia

Good roads at any price seems to be the motto in many parts of the country and the worst of it is that adequate measures are not always taken to make sure that the roads are really good. No longer ago than Aug. 7 of this very year A. D. Williams, chief state road engineer of West Virginia, wrote a letter to "the Honorable County Court and County Engineer" of a county of that state in which he gave figures indicating that by the exercise of more care and skill in the design, specifications and contract for some three miles of road improvement the contract price might have been cut by $36,070, or from $79,270 to $43,200. Some of the possible saving claimed was based on "a letter from a responsible paving contractor" who wrote the governor that if penitentiary labor were provided for grading, as the governor had suggested, he (the contractor) stood ready to pave the three miles of road for $30,000 or less, which appears to be about $23,500 lower than the contract price on the corresponding part of the whole contract. Obviously, this sort of an estimate is less weighty than an actual competitive bid, besides not taking into account that the convict labor can hardly be classed as all clear saving. The other figures in Mr. William's estimate are more specific and while we make no pretence of having checked them in the light of local conditions they certainly seem to indicate that thousands of dollars could have been saved by better bridge and culvert design, cutting out unnecessary retaining wall, and giving more careful attention to overhaul. Unfortunately the contract had already been let before the state road engineer submitted his figures, so they can serve only as a warning for the future.

Mr. Williams points out that as there are 32,000 miles of roads in West Virginia, contracts at the rate of over $26,000 a mile would mean some $845,000,000 "to construct the roads of this state, which is an impossible figure." This is, of course, a mere *reductio ad absurdam* since even the most reckless good roads advocate would hardly propose to cover the whole 32,000 miles of roadway with a high-grade surface, no matter how economically the work were to be done. All the more need, therefore, for careful choice of types of road surfaces and all accessory construction and for the best of engineering design and supervision throughout, in order that whatever sums are available may be made to go as far as possible.

Credit is due the Governor of West Virginia (A. D. Hatfield) for the stand he has taken in connection with this road incident. He has made it known that while he favors widespread road improvement he recognizes the necessity of good engineering and of economy. May governors of like mind increase in numbers!

⁂

The True Value of Public-Utility Property as it Affects Public Welfare

In our issue of July 31, last, we noted the main points of a recent decision of the Appellate Division of the New York Supreme Court (in the case of the Kings County Gas Co. vs. Public Service Commission for the First District of New York) which seems practically to order the state public service commissions to include "going value" of a public-utility property (over and above development cost) and appreciated value of street mains (due to laying improved paving since the lines were put down) in any basis for rate making. The court also refused to allow annual increments of value to be entered among receipts to offset the burden of depreciation which must be met out of earnings.

Reference to ENGINEERING NEWS of Aug. 31 and Nov. 16, 1911, shows a summary of the Commission's reasons for refusing to accept going value* and for rejecting the plea of increased value for buried pipe. From the viewpoint of common equity and public welfare the Commission, we think, was justified in its stand, nor does it appear that any hardship was imposed on the corporation—at least none if we accept the principle of adjusting the

*In the case of the Kings County Gas Co., it was desired to have entered in "going value" $90,000 annual profits from favorable street-lighting contracts, capitalized at 4½%. As "fair return," 10% was asked which would make these contracts yield some $200,000 per year—$110,000 coming from general service outside of street lighting.

rates to yield a fair return on the amount actually invested in its business. The findings of the learned court in this case may be plausible law, but they do not seem to be in accord with social or economic justice; they must be included among those decisions whose tendency is to create and enthrone property at the expense of general welfare.

The court holds that valuation for purchase must be essentially the same as for rate making and that if going value is acceptable in one case it should be in the other. That does not seem necessarily so always. Suppose a company holding extra-profitable contracts for street lighting wishes to sell its business; the probability is that the purchaser would be willing to increase the purchase price on account of those expected unusual profits, paying a bonus which would cut down his expected earning to but little more than what the old company could expect to get without fat contracts. Does the purchaser expect, then, that he can include that bonus in the value on which *all* his future rates are to be based? It is hardly conceivable that he expects to boost all of his gross earnings just because at the moment one department is yielding an uncommonly large profit on actual investment.

This bonus is given for added earning power which is temporary in the eyes of the purchaser, due to the limited term of the contracts. Depending on the wording of the contracts as to effect of accidents, tornados, floods, etc., such a bonus has more or less the nature of a gamble. At any rate, to allow the capitalization of such temporary annual profits, even at the same rate which the property is allowed to earn, injects a false element of "value" on which the general customers may have to pay when the favorable contracts lapse.

The court's second proposition in this case, to allow the company revenue on unearned increment in the value of its buried mains but not to allow annual value of increment to help relieve the burdens of depreciation, is peculiar though perhaps it is good law. The company necessarily has been passive while the community has been active. The mains are more expensive to replace in proportion to the improvement wrought by the city. The existance of absolute and unqualified rights to such peculiar property value may be questioned; has the community no interest in it? The court seems to intimate that it is equivalent to confiscation of property not to make this unearned increment of value bring in its 6%. It would have seemed a logical parallel to this view that the annual increment would have entered the accounts some way to help balance depreciation. But it is held that such increments of property in service cannot be used for payment of obligations and hence cannot be entered as receipts. It might be advisable, then, to consider setting up "appreciation accounts" similar to the present "depreciation reserves," or to treat depreciation in its negative as well as its positive sense.

Whether the commission will take an appeal from this decision has not been announced. The principles at stake are of such great importance to the people of the state that, in the hope of securing a closer approach to justice, a ruling by the highest court should be secured. If the principles laid down by the recent decision are sustained, then a closer definition of the basis for adjusting rates should be included in future public-utility franchises—a thing, however, which is desirable in any event.

On Self-Appointed and Popular Elected Municipal Experts

In welcoming the American Society of Civil Engineers to Ottawa at the recent annual convention, the mayor of the city laughingly said that every second man in Ottawa considers himself a water-works expert. It is always well to see the humorous side of a situation but water-supply conditions at Ottawa during the past five years have not been a matter for merriment. There have been two bad outbreaks of typhoid fever due to defective water-supply mains leading from the river intake to the pumping station. Plans have been prepared both for a new gravity supply for the city and for purifying the present supply, but each scheme was disapproved by popular vote. Since these adverse votes, Sir Alexander Binnie and Dr. A. C. Houston have been brought to Ottawa from London at large expense to advise the city government what should be done. They have recommended still another and much more distant source of supply and surveys are now being made by a representative of Mr. Binnie to determine the conditions governing the supply recommended by him.

Only a few days ago a lower court confirmed an award of various allowances totaling some $13,000 made by the Ottawa city authorities to a number of persons and firms who were employed to fix responsibility for the second typhoid outbreak. It is said that among these experts was a stone mason who was paid $50 a day for a considerable period. Although Ottawa has gone to a large expense to obtain advice on water-supply matters, thus far to no effect, and although much of this advice has been from competent sources, every other man in the street seems to have qualified himself, to his own satisfaction, as better able to judge as to the best water-works plan for Ottawa than the men who have made water-supply their life study.

It appears to us that what Ottawa most needs is not that a multitude of its own citizens shall appoint themselves experts on the city's requirements but efficient civic leadership. The city government appears to be in that muddled condition so common in many cities where the powers and responsibilities of the different branches of the government are not sufficiently centralized and correlated to assure firmness and certainty of decision and execution. With the leadership of one strong, efficient man, be he called a boss or a leader, progress might be made under a poor system of government; but without such leadership, no matter how good the scheme of government, factions arise and work against each other and succeed only in preventing progress. Under such conditions, even if the city government decides on a given course, the electorate is likely to vote against it, as has happened at Ottawa in water-supply matters.

Differences in popular opinion as to this or that source of supply or method of purification seem to have been largely responsible for lack of definite progress in deciding upon some one of the several projects for better water at Ottawa. There seem to have been two factors which indirectly have contributed to create the conflicting popular opinions. One of these is the residence in Ottawa of a considerable number of technical men connected with the Dominion government, some of whom might have official authority in the matter of water-supply if Ottawa were under Dominion control as the city of

Washington is under Federal control in this country. The other factor which has probably contributed more or less directly to the defeat of the two projects already voted down is that each has been worked up by a New York engineer. We note this popular prejudice merely to express regret that in such professional matters there should be any such thing as an international barrier to the spread of engineering practice from either side across the boundary.

Neither the electorate, the city officials nor the framework of the government of Ottawa is any worse as regards the meddling of self-constituted experts, lack of leadership or lack of clear-cut division of powers and full centering of responsibility than is to be found in many cities of the United States. The cities of neither country have as yet "found themselves," in a governmental way. Moreover, just as the cities of each country, and particularly those of the United States, were experimenting with various types of city government from which improvement was hoped, a wave of what might be termed ultra-democracy came sweeping from the Pacific to the Atlantic. With it representative government has been either swept away or so deeply submerged as not to be able to stand steadily on its feet.

Big and virtuous with a feeling of return to a power which it had lost for a while through its own neglect, the general electorate is no longer content to choose men to represent it but is determined as well to settle all matters of policy and even minor details. Thus we have the initiative and the referendum, each good when used in moderation but liable to abuse.

This is not all. Under the commission plan of government, which combines the legislative and executive functions of the whole city in five men, the people elect executive officers, many of whom must carry on or try to conduct highly technical work. Experts can no more be created by popular vote than they can be self-elected. Men chosen by popular vote to technical positions come to feel, sooner or later, that they are experts, regardless of lack of education and experience. The result of this sisterhood of self-created and popularly elected experts, when combined with a muddled city charter, is poor municipal government, sometimes accompanied by typhoid epidemics and by indecision over means to avert them, as at Ottawa—although we do not mean to imply that Ottawa has commission government.

The remedy should be obvious to anyone, and especially to engineers. Our schemes of city governments should be designed first of all for the efficient execution of municipal work. The electorate should choose a few honest, capable men to formulate all policies—unless it be a few of sufficient breadth to be determined in outline by the voters themselves. The *execution* of these policies should be entrusted to experts in the various branches of administrative government, especially in the highly technical fields; and these experts should be *appointed* by the chosen representatives of the people and not elected by the people themselves. When unusual problems arise, outside experts should be called in. As a rule the advice thus secured should be followed by the responsible executive officers. Obviously such a program leaves no place for the self-appointed expert, especially when he becomes so numerous as to equal half of the electorate of a city.

Making Membership in an Engineering Society a Tangible Asset

There is an engineering society in Great Britain, which, unfortunately for the younger members of the engineering profession in America, has no counterpart in this country. This society is the Junior Institution of Engineers. It was founded primarily, as its name implies, for the benefit of young engineers; but since the Junior Institution of Engineers has been established many years its membership now consists not only of real "juniors" but of numerous former "juniors" who have now reached the top-notch positions in the engineering and business world. These men appear to retain an uncommon loyalty to the Institution, and justly so, no doubt, from a sense of gratitude for the encouragement and inspiration it gave them during the hard-knock periods of their experience.

The nature of this society permits it to extend its activities beyond the narrow limits of the more distinguished and scientific Institution of Civil Engineers or the Institution of Mechanical Engineers, of which many of the older "juniors" are also members; or of our own distinguished American Society of Civil Engineers.

One of the objects of the Junior Institution of Engineers, like all other engineering societies, is to advance the knowledge of engineering by the presentation of original papers and discussions at regular intervals; but the Junior Institution has other meetings where the members have a much better opportunity to become acquainted with one another. It has banquets, balls, teas, garden parties and other social gatherings, where members meet on a common and friendly footing; not only members but the wives, sisters, mothers and sweethearts of members as well. It is the nearest to a *real fraternity* of engineers of which we have any knowledge.

The Junior Institution has a monthly *Journal*—not wholly of the dignified type so common to engineering societies in this country, but a real live periodical containing besides the professional papers and discussions of the society extracts from letters of members and former members in all the four corners of the globe—written as familiarly as one friend writes to another. The *Journal* also publishes personal items regarding the movements and work of the members, and often little editorial notes and addresses from the older and distinguished members, with much sage advice to the younger and inexperienced members.

This brief outline of the Institution's activities alone will interest every young civil engineer in America who has had some active experience in railway location and construction work, or on any large construction project, where he has had an opportunity to see and enjoy the fraternity and feeling of common interest which seem so naturally to bind together those who follow the profession of engineering.

Recently, however, the Junior Institution of Engineers has taken another step of still deeper interest to the young engineers of America. This step is the establishment of a permanent Engineers' Register, in which it is hoped a complete record of the professional career of every member will be filed. The express purpose of the Engineers' Register is to form an engineering clearing house, to assist members in improving their positions by systematically recording the particulars of

each member's experience and technical training, and by effecting introductions to engineering employers of those "juniors" who possess such qualifications as may fit them for vacant positions. More than this, however, since many members are, as noted before, no longer juniors in point of age and experience but are practicing today as consulting engineers or in other private work, it is proposed to make it a part of the duty of the Engineers' Register to see that these engineers receive information of possible business and introductions to possible clients.

The Engineers' Register will, of course, involve a large amount of work, but such is the loyalty and zeal of the members of this society that a special committee has been for some time enthusiastically at work on the details of the Register, and the scheme it has adopted will be followed out in full, taking the place of an "appointments board" which has been maintained for several years.

In accordance with this plan the secretary sent out, a few months ago, blank cards to be filled in by the members, which were to be for the private information of the secretary and the chairman of Engineers' Register committee. This experiment soon proved the worth of the Register. Considerable numbers of these cards have been returned and classified and the whole scheme established on a business-like basis.

All the members who have applied to the Engineers' Register so far have obtained satisfactory appointments and no applicant is now (July 1) waiting for employment. Furthermore, since the establishment of the Engineers' Register the secretary has received applications from employers in greater numbers than he has applications for employment.

In the July issue of the Junior Institution's *Journal*, the chairman of the Engineers' Register committee has this to say:

> It has been thought desirable to again call members' attention to the scheme and to request their interest in what must be an extremely valuable advantage, not only to individual members, but also to the Institution as a body. It is only by the coöperation of all that the fullest benefit can be obtained, and it should be remembered that it is not merely intended that the Engineers' Register shall help those who happen to be without employment, but also those who, although in regular work, are looking out for a chance to improve their positions.
>
> It is an undoubted fact that an engineer can obtain higher remuneration for his services when a position is offered to him than when circumstances force him to compete with others. Immediately he is in the unfortunate position of being unemployed, his earning capacity is depreciated by perhaps 50%. It is with this knowledge that the Council has instituted the Engineers' Register. Negotiations for future employment can therefore be commenced before necessity compels the acceptance of a lower wage than would be earned by the same man if introduced to an employer while still engaged in his profession.

The blank forms sent out to the members with the July issue of the JOURNAL are four-page folders, 8½x10½ in. The first page contains a letter, in part as follows:

> The Engineers' Register, recently reorganized and put into practice by the Council of the Institution, having proved so beneficial in obtaining appointments and advancement for the members, it has now been found advisable to classify those members who wish to avail themselves of the scheme. Primarily the object of this Register is to obtain for members more advanced positions in the engineering profession, but as the Secretary of the Institution is frequently asked for information regarding persons who have special knowledge in a particular branch of the profession, it is hoped that all members will fill in and return the attached form, so as to make the Register as complete as possible.

The blank form has ten main divisions or classifications of engineering practice: general, which includes teacher, draftsman, consulting engineer and similar broad classifications; civil, mechanical, electrical, shipbuilding, power generation and transmission, manufacture, transit and communication, miscellaneous, and one blank left for filling in any specialty not listed. Under each of the main classifications are ten subdivisions or blanks, making approximately 100 specialties for as many classifications. The remaining two pages of the blank form are to be filled out by giving the particulars of training, experience, etc.

Upon the return of these forms to the secretary it is proposed that the particulars furnished by the members shall be rewritten in appropriate form by the secretary, omitting the name and address of the applicant, and these specifications sent to firms which may at some time or other require the services of such men. For this purpose the special committee having this matter in charge has in preparation a classified list of employers, compiled from information supplied by personal knowledge and by the aid of other members of the Institution. If a member so desires an advertisement will also be inserted in the "Blue Circular" issued monthly with the *Journal*. All such services are, of course, free to members.

Here, then, is an engineering society where membership is a tangible asset; furthermore, a society which breeds loyalty and enthusiasm in its members.

While regretting that the younger engineers of this country have not a similar institution, we do not believe it is by any means necessary to start a new society where engineering societies are already too numerous. But we do believe the work and activities of the Junior Institution of Engineers are worthy of the study of those engineers in this country who believe an engineering society can and should be made of tangible benefit to its members, to the engineering profession—and incidentally to the public at large by placing the right men in the right places, and keeping out of the profession the dishonest, inefficient and unfit.

It is worth remembering that the Junior Institution of Engineers is in England, which is sometimes thought of by Americans as unprogressive in engineering. It may also be remembered that so conservative a society as the Institution of Civil Engineers has had an "Appointments Board" for some two or more years.

Turning to the progress made in this country on similar lines we find in our issue of May 8, 1913, p. 974, a letter by a prominent member of the American Society of Civil Engineers in which is the following comment:

> It is truthfully said that the only benefits of membership in the American Society of Civil Engineers is the receipt of a few publications, the value of which is yearly deteriorating, and the honor connected with membership.

Another member, in our issue of Jan. 2, 1913, in emphasizing the need of more concerted action on the *human* side of engineering wrote:

> As the American Society of Civil Engineers is already in the field with a large membership and an effective organization, it would logically seem to be the proper body to undertake this work. If, however, it should decide that it is unwise for it to undertake such action, then a new organization is needed, and will undoubtedly develop.

We have taken the American Society of Civil Engineers as an illustration because it is the oldest, largest, richest and most influential of our great national engineering societies—the one which would naturally be

looked to for taking the initiative in all matters pertaining to the benefit of the engineering profession—and because many of its members have already shown a desire to broaden its influence and usefulness.

Careful readers of our columns during the past two or three years cannot fail to see how very deep and widespread is the interest with which engineers are following the discussion of the problems affecting the *human* side of engineering—the status of the engineer, his compensation and all those problems which affect him as a breadwinner and as a man among men. The overcrowding of the profession will make these problems more pressing.

Many of the younger members of the profession are becoming restless and dissatisfied with the attitude of those who control the work and activities of the engineering societies. Under these conditions it would seem that no opportunity should be lost by a great and influential society, whose real purpose is the betterment of the engineering profession, to emphasize the brotherhood and community of interests which are essential to a loyal and zealous support of any society.

It is, of course, true that the larger a society becomes the harder it is to handle this human element feature, but even a very large society can do a great deal to win and keep the loyal support of every member. And one of these means is the establishment of a bureau or department where any member can, if the occasion requires, market his special ability to the best possible advantage, as similar English societies are already doing.

Aside from the help an Engineers' Register offers to those seeking employment the scheme has other highly desirable features. It is not designed to encourage the young engineer to change his position frequently, as might appear at first glance. Instead, it offers every inducement to him to stick to one occupation and make a reputation for himself and his work. It gives him a feeling of certainty that if he can develop particular ability in some line of engineering work he can find a place to market that ability to the best advantage. It gives him a feeling that he has a powerful and interested friend ready to help him if he proves worthy.

Is not this a great inducement to honest and efficient work in all branches of engineering? And would it not go a long way toward quieting the unrest of engineering employees and the ever-present agitation for licensing engineers, and in some quarters of the secret agitation for labor-union methods of improving the condition of young engineers?

LETTERS TO THE EDITOR

The Uplift Pressure in Dams

Sir—The article in your issue of July 31, 1913, p. 202, by C. R. Weidner, giving the results of tests of upward pressure on dams in Europe, is of great importance, in the writer's judgment. This is not because the results of these tests are in any way startling, but merely because, at great expense, a thing has been demonstrated that any logical mind familiar with the principles of mechanics should have reasoned out.

With the exception of one very recent book, the writer believes that no English work on dams makes the slightest mention of upward pressure as a force that should be considered in designing dams.

The results of these experiments further emphasizes what the writer has before said: *It is a crime to design a dam without considering upward pressure.*

Some months ago (ENGINEERING NEWS, Jan. 16, 1913) I criticized the design of a hollow dam, because upward pressure was not considered. The designer pointed out that he had considered 30 ft. of "backwater." He did not dwell upon the fact that he had counted upon the full "back pressure" of this water as well as its vertical downward pressure on the slope of the dam. Also he acknowledged that with full upward pressure at the heel tapering to zero at the toe, there would be a tension at the heel, which *could be* "easily provided for" (There was no such provision in the design).

I cannot conceive of the condition that would demand a dam 100 ft. high, where the tail water is 30 ft. deep. This is the idea many designers and authors have of upward pressure, namely, that there must be water on both sides of the dam in order to give any upward pressure whatever. They will acknowledge that if a stone is submerged it will lose weight in proportion to the water displaced, but they fail to see that if a stone is laid against a body of water, the pressure of that water forces it underneath the stone and the upward pressure exerted on the bottom of the stone helps to overturn it.

It is the simplest thing in the world that water should be found to exert pressure under a dam just about as the experiments described by Mr. Weidner. If a hole be drilled almost anywhere near a body of water and cased to the bottom, water will rise in the hole to the level of the body of water. If the water entering the hole at the bottom were not under pressure, it could not rise in the hole.

EDWARD GODFREY.

Pittsburgh, Penn., Aug. 11, 1913.

The Cables of the Halsted St. Lift Bridge at Chicago

Sir—You will be glad to publish a correction of a statement made on p. 920 of your issue of May 19, in regard to the repairs to the South Halsted Street lift bridge over the Chicago River. The statement with regard to the service of cables is incorrect. Engineers of the city of Chicago had formerly advised us (and this has since been verified) that the main suspending cables of the lift bridge, consisting of eight 1½-in. plow-steel cables at each corner of the span, have never been renewed, but the original cables placed when the bridge was built about 20 years ago are still in service.

Previous advices, also, were to the effect that the

operating cables have been renewed twice; once several years ago and again in 1912. It is possible that there may have been some mistake in the nomenclature, as some of the operating cables are attached to the counterweights and may have been referred to as counterweight cables. The principal fact of interest, however, is that the main suspending cables carrying the total load of the lift bridge are still in service after about 20 years of use. I am sure that you will be able to verify these statements, and so to make correction.

Information on the life of wire cables under service is somewhat meager, and it seems undesirable to have erroneous statements uncorrected in engineering literature. It may be of interest to users of cables to know that the Bureau of Standards of the Department of Commerce and Labor is undertaking experiments on the life and action of cables operating over sheaves. The most extensive experiments of that nature which I have come upon in the publications of the Institute of Civil Engineers indicate a probable life for cables operating under conditions of the Halsted Street suspending cables in excess of 20 years, so that the correct recital of the conditions there bears out the conclusions from the experiments.

In connection with the cost figures given, it would be of considerable interest if you could give a division of the costs of maintenance and repairs, advising how much is paid to the operators, to mechanics and others employed regularly on the bridge; how much of the amounts given are for general maintenance, pavement, structural work, etc., and how much for the actual operation of the machinery and mechanical parts, for oil, waste and items pertaining directly to the mechanical operation of the span and not to the maintenance of the bridge as a whole.

It is possible, also, that you may be able to give similar information for other movable bridges operating under the same conditions and with the same frequency as the Halsted Street lift bridge, which, like it, were designed 20 years ago. Such comparative data would be of interest to those interested in the development of movable bridges.

Very sincerely yours,
WADDELL & HARRINGTON,
Per E. E. Howard, Asst. Engr.

Kansas City, Mo., July 2, 1913.

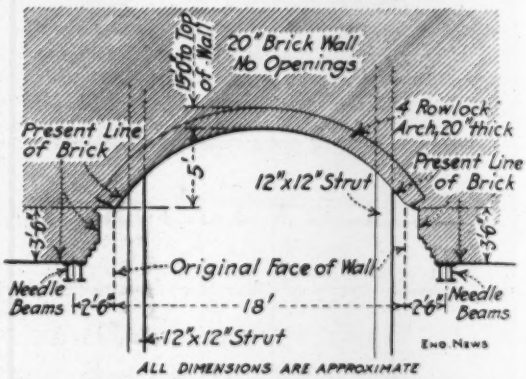
[We learn from Mr. Hammond, Engineer of Bridges and Harbor, Chicago, that the counterweight cables have never been renewed, while the operating cables were renewed last year and also once previously. The annual expense for operators is $2400 and for electric power $600, making a total annual expense of $3000, exclusive of a direct charge of repairs and maintenance. He is not prepared at this time to give in detail the itemized costs suggested in the above letter. Possibly some of our readers can furnish such information in regard to other bridges.—EDITOR.]

A Question of Stability

Sir—The following account of how a large brick arch is standing at the present time, with parts of its abutments entirely cut away, is a very good illustration of the remark once made by a great engineer, who, in response to a query as to why a certain structure stood up, replied "By the Grace of God and the force of habit," and for this, if for no other reason, the following description cannot fail to be of interest to all engineers and contractors.

Extensive alterations are being made to a church situated on the west side of the City of New York, a few blocks north of 100th St. These alterations, inso far as they have been carried out up to the present time, consist in entirely wrecking one wing of the church, in order to make way for a new addition. The wrecked wing was separated from the main body of the church by a 20-in. brick-bearing wall, which wall has been left in position, except as hereinafter described. Communication between the two parts of the edifice through this wall was by means of a large opening about 18 ft. wide, spanned by a four rowlock brick arch. This arch not only carries the 20-in. wall extending for about 15 ft. above the crown of the arch, but also carries a small portion of the roof of the main part of the church.

Part of the new work to be installed consists in placing steel columns in this wall, and in order to do this, the



A BRICK ARCH WITH ABUTMENTS REDUCED IN COURSE OF REPAIRS TO BUILDING

contractor has braced up part of the roof of the main building resting on the arch with long 12x12-in. strut timbers extending to the main floor of the church, has run needle beams through the wall at various points, two sets of which are shown in the accompanying sketch and then has cut away the wall under the needle beams, and also under the ends of the brick arch. This is all clearly shown in the sketch, where the original face of the wall is indicated, and the present line of the brickwork shown in a heavy line.

Why this arch stands, is something of a mystery, but the explanation, as far as any can be given, is as follows: The brickwork above the arch is acting as a false arch and aided by the temporary struts supporting the roof, is carrying the load above, the true arch really being supported by this false arch, and the bricks of the arch are held in position solely by the adhesion of the mortar. It is a question in the writer's mind how far such a risk should be taken, with a span of this size, even with first-class brickwork. He strongly condemns this particular case, and indeed expects any day, to pass by and find this arch in ruins. It is such work as this that when a failure occurs, brings engineers and engineering into disrepute, although if the facts were known, it would probably be found that no engineer had anything whatever to do with this particular piece of work.

J. H. GANDOLFO.

43 Exchange Place, New York City, Aug. 7, 1913.

Automatic Sprinklers as Life Savers

Sir:—The writer has noted the editorial in your issue of July 31, referring to the Binghamton fire disaster. It strikes me that one of the strongest points you could have made has been missed in that you did not call attention to the use of the automatic sprinkler, as the means of preventing both fire and panic, and, therefore, of safeguarding lives in a more effective way than could possibly be done by any means of merely getting people out of the building. The fact that the sprinklers would, at the same time, completely safeguard both the building and its contents, as well as the lives, cannot be lightly regarded.

In the August number of the *Journal* of the American Society of Mechanical Engineers, the case is very well put on pages 1272 and 1273, as well as, in a certain sense, the remarks at the bottom of page 1269 and top of 1270. Particularly in the middle of page 1272 is the point made clear, that every time a sprinkler operates from fire in a building full of people, it is a potential life saver.

The writer has been studying this subject of automatic sprinklers, and their use in saving both property and life, for some little time. So far as I have been able to learn, there have been only eight deaths by fire in factories protected by automatic sprinklers. In not a single one of the eight cases could the death be charged to the failure of the sprinklers to operate properly, as you will see by considering the details.

In January, 1907, under peculiar circumstances, a fire gained great headway in the Cocheco Mills, Dover, N. H., at a time when the automatic sprinkler system had been temporarily shut off for adjustment. This was a cotton mill, and the spread of the fire was so rapid in the cotton goods on the machines, and up through belt ways, etc., that five of the employees, who attempted to save their belongings in their lockers, were overcome by smoke, and lost their lives. The sprinklers were not operative at the time the fire broke out. Before they could be adjusted and put in service, the damage had been done. This is, of course, a strong argument in favor of keeping all such equipment, at all times, ready for immediate service.

About a year ago, a celluloid comb factory, near Boston, was the scene of a fire which flashed all around the room in scraps of extremely inflammable celluloid on the work benches, making a complete circle of flame, which shut off access to the stairs. About a dozen employees in the room dashed through the flame and down the stairs, all being more or less severely burned, and two of them losing their lives. But, please note, before the fire department arrived on the scene, the automatic sprinklers had put the fire *completely out*. It would, of course, be too much to have expected the employees to remain within the circle of flame for the 60 or 90 seconds required by the sprinklers to put out the fire. Probably no human being, under such circumstances, and with the panic which is always conected with fire, would have remained at this time. But it is perfectly clear that if all the employees had remained in the room, the chances are that none of them would have been severely injured.

The other case, where life was lost, under automatic sprinklers, was in a piano factory in New Jersey, also about a year ago. The watchman's lantern exploded, while he was making his rounds, and set fire to the inflammable varnishes in the room through which he was passing. He was so severely burned, as a result of the initial explosion, that he died the next day. The sprinklers held the fire from spreading, giving the firemen an easy task in putting the fire out.

Practically every expert who reported upon the results of the Asch Building fire in New York two years ago, said that automatic sprinklers would probably have prevented the loss of life in that building. As you have probably noted, automatic sprinklers have been since installed throughout that building.

Cases are on record where a fire in a waste basket full of paper has been put out by a sprinkler before the waste basket was consumed. The sprinklers operate without human assistance, are not subject to panic conditions or excitement, are not hampered by smoke or a sudden flash of flame, are not hindered in their work by a rush of frightened people from the seat of disturbance.

To my mind, they are the only great saving factor in our fight against fire, and should be required (legally) in all manufacturing plants subject to destruction. They can be put into existing buildings at nominal expense, and render what is now a fire-trap, more safe as against both panic and loss of life, than any "fireproof" building now existing, which does not contain sprinklers. If you consult the real estate advertising pages of the New York *Herald*, you will find building after building designed for loft and manufacturing purposes making capital of the fact that they are "protected by automatic sprinklers."

SIDNEY G. KOON
Mechanical Engineer, Walter B. Snow, 170 Summer St., Boston, Mass., Aug. 15, 1913.

⌘

The Railway Requirements for British South Africa are greater at the present time than ever before, according to the "British and South African Export Gazette" (London, England). New locomotives to the number of 113 will be required. All except eight of these are to be equipped with superheaters of the "Schmidt" type. Most are of special design according to the several peculiarities of the country, from heavy types for the Orange Free State, to side-tank engines of 2-ft. gage for some of the branch lines. The bulk of the orders have already been placed, but bids for others are still to be requested.

⌘

A New Type of Concrete Pile, which would seem to have little merit except its novelty, has been put on the market by the Siegwart Company, of Lucerne, Switzerland. The new pile utilizes the bulged bottom principle common to several patented piles, but is novel in that the enlarged base is formed by the use of explosives. The process of making the pile is described as follows:

An iron tube containing a wooden pile with an iron point, and a device at the upper end for deadening the blows of the pile driver, is driven into the ground. The wooden pile is then removed and the cartridge provided with an electrical detonator is fixed in position at the bottom of the hole, and the tube is filled with wet concrete, a little above the level of the surface of the ground outside the tube. The iron tube is then drawn up some feet to prevent its end from being shattered by the force of explosion. After the detonation the concrete sinks downwards, filling up the space hollowed out by the explosion. Finally the tube is little by little filled with more concrete, and is drawn up gradually at the same rate as the concrete is filled into it.

After the completion of this process, and after sufficient time has been allowed for the hardening of the concrete, there is in position in the ground a pile with a base of greater lateral diameter than it is possible to produce by any other means. Moreover, on account of the action of the gas formed by explosion, these piles will give the best possible results in compressing the earth. In fact, one pile on this system serves the purpose of several piles on any other system, thus insuring a great economy. By using cartridges of greater or less strength the size of the excavation made by the gas is controlled at will; by the use of a cartridge 600 to 700 grams in weight there is for instance made a spherical hollow of 3 to 4 ft. in diameter, according to the quality of the ground material.

A Fire, Load and Water Test upon Cinder-Concrete, Terra-Cotta and Gypsum Floor Arches*

BY HAROLD PERRINE†

The system to be tested forms the roof of the test house, which structure is essentially a cinder-concrete oven of permanent construction. Properly supported upon piers 2½ ft. from the ground is a grate composed of railroad rails and wire mesh, about 280 sq.ft. in area. The vertical distance from the grate to the roof is about 9½ ft. The floor system to be tested is supported by steel I-beams resting on the tops of the walls, spaced at distances suitable to the span or spans under test, and having a clear span themselves of about 14 ft. Ample draft openings and flues are provided to facilitate the control of the fire.

The structure, as well as the methods used in the herein described test, conform with the specifications adopted by the Bureau of Buildings of the City of New York and by the American Society for Testing Materials.

The floor system under test was comprised of three separate and distinct types of arches, i.e., cinder concrete, terra cotta and gypsum.

In the first or west bay (Fig. 1) was installed a 4-in. slab, 3 ft. 3 in. in length, of one part portland cement, 2 parts Cow

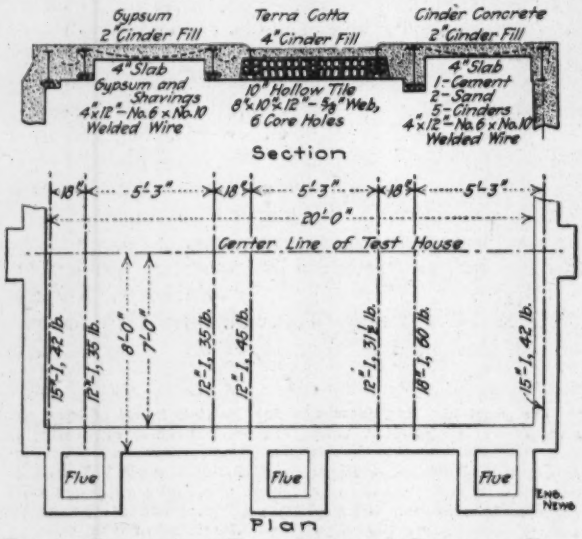


FIG. 1. DETAILS OF TEST SLABS AT COLUMBIA FIRE-TESTING STATION

Bay sand and 5 parts clean hard-coal steam cinders, mixed medium wet, reinforced with Clinton electrically welded wire reinforcement, 4x12 in., No. 6 x No. 10, of 75,700 lb. per sq.in. tensile strength, every sixth wire clinched over beam flange, covered with a 2-in. cinder fill. Age on day of test, 29 days.

In the second bay was built a side construction terra-cotta arch 5 ft. 3 in. in length, of "Natco" 10-tile, 8x10x12-in., 6-hole, ⅝-in. web laid up wet in 1:3 portland-cement mortar, fairly well grouted between blocks, but with no mortar between skew-backs and surfaces of supporting I-beams, all covered above with a 4-in. cinder fill. Age on day of test, 29 days.

The third 5-ft. 3-in. span was composed of a 4-in. slab of a mixture of gypsum and wood shavings, furnished by the U. S. Gypsum Co., reinforced with identical type of Clinton wire mentioned above, every wire clinched over beam flanges and covered with a 2-in. cinder fill. Age on day of test, 14 days.

Each supporting beam was protected by the material comprising the adjoining arch.

One-half the under surface of each of the first two arches was plastered with two coats while the gypsum slab was plastered throughout its entire length (Figs. 5-6).

The estimated weight, dry, of the cinder slab material is 98 lb. per cu.ft. The estimated weight, dry, of the gypsum slab material is 70 lb. per cu.ft.

*A report on a test made for Albert Oliver, of the Clinton Wire Cloth Co., 103 Park Ave., New York City, at the Columbia Fire Testing Station, Greenpoint, Brooklyn, N. Y., July 29-Aug. 1, 1913.

†Assistant, Department of Civil Engineering, Columbia University, New York City.

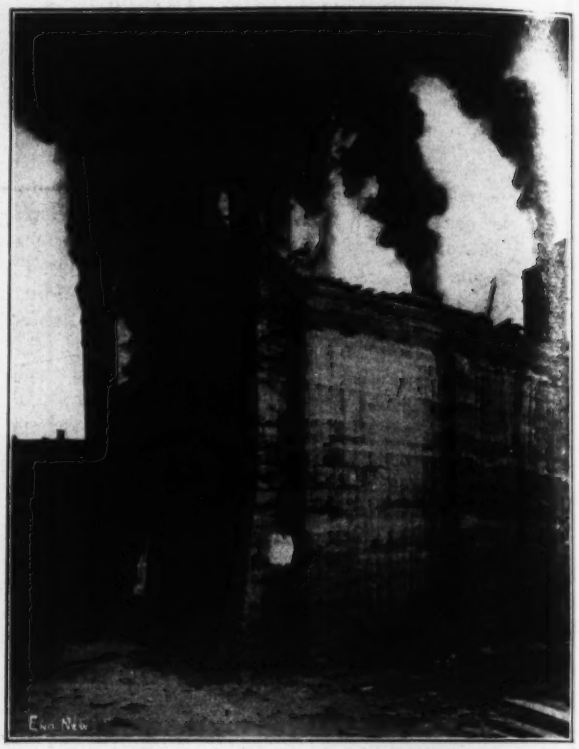


FIG. 2. VIEW OF TEST HOUSE IN ACTION, ALL FLUES ACTIVE

The purpose of the test was to determine the effect of a continuous fire below the floor lasting four hours, at an average temperature of 1700° F., a temperature generally conceded to be that of a burning building, the floor carrying at the same time a distributed load of 150 lb. per sq.ft. At the end of the four hours the under side of the floor, while still red hot, was subjected to a 1⅛-in. stream of cold water through a hose at short range, under a 60-lb. pressure, for 5 min.; the upper side of the floor was then flooded with water

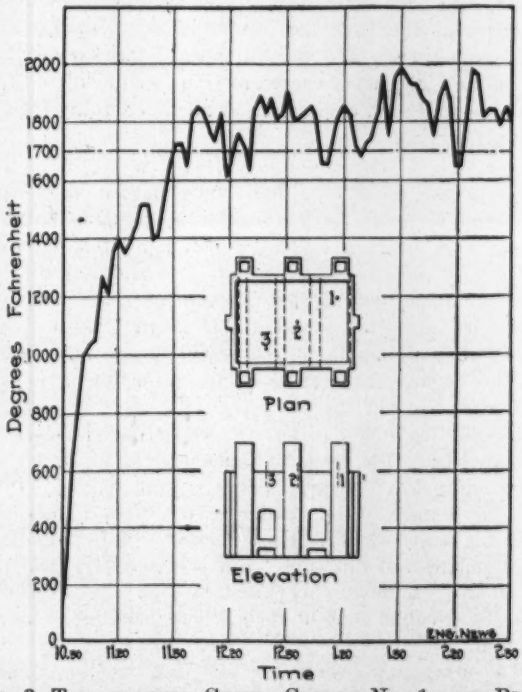


FIG. 3. TEMPERATURE CURVE, COUPLE NO. 1, AND PLAN SHOWING POSITION OF THERMOCOUPLES

at low pressure, and afterward the stream applied at full pressure to the under side for 5 min. longer. Deflections of beams and floors were measured continuously during the test. On the following day, when the floor was cool, the load was increased to 600 lb. per sq.ft. and deflections noted.

LOAD—Pig iron, stacked in segregated piles to eliminate arching, was the material used for load.

During the fire the cinder concrete and terra cotta arches carried 150 lb. per sq.ft., but the gypsum only 75 lb. per sq.ft.

FIG. 4. UNDER SIDE OF CINDER-CONCRETE SLAB AFTER THE FIRE

TEMPERATURE—The temperature of the fire was measured by three electric pyrometer couples suspended through the floor from above and hanging about 6 in. below the ceiling. The locations of the couples are indicated on the plan of the building. Readings were made upon each couple every three minutes. The fuel used was dry cord wood, one-half oak and one-half pine, the frequency of firing being determined by the temperature of the test chamber. Temperatures are noted in Fig. 3.

DEFLECTIONS—Deflections which occurred during the test were measured by a Y-level reading upon rods located at the ends and middle of each beam and at points over the centers of the slabs. (See accompanying table).

CORRECTED DEFLECTIONS FOR MID-POINTS OF ARCHES

	Cinder concrete in.	Terra cotta in.	Gypsum	in.
End Fire				
150 lb. per sq.ft. load..	1/8	1 1/16	75 lb. per sq.ft. load.....	1/16
Floor Cool				
150 lb. per sq.ft.....	1/8	1/8	75 lb. per sq.ft. load.....	1/16
Total Load			Load 450 lb. per sq.ft.....	1/8
600 lb. per sq.ft.....	3/16	1/4	Load 496 lb. per sq.ft. Failure	

WATER—Water was applied by firemen with an engine detailed from fire Station No. 238, Greenpoint, Brooklyn. A pressure of well over 60 lb. was maintained at the nozzle. The stream was thrown back and forth over the ceiling and not allowed to strike continuously in one spot. The total time of the two applications at full pressure was 10 min.

GENERAL OBSERVATIONS—Day, fair; temperature, 95° F.; age of cinder and terra-cotta floors, 29 days; age of gypsum arch, 14 days.

EFFECT OF WORKING LOAD—None of the arches showed appreciable deflection under initial load, i.e., cinder concrete and terra cotta, 150 lb. per sq.ft., gypsum 75 lb. per sq.ft.

EFFECT OF FIRE AND WATER

CINDER CONCRETE—The combined effect of fire and water was to remove the plaster entirely from the slab, leaving the concrete itself in excellent condition. It was only in a region near its center, in order to strike which, the stream had to be elevated to nearly a vertical position, and consequently causing maximum punishment that the surface was slightly pitted (Fig. 4).

The protection to the deep supporting I-beam naturally suffered more severely. Possibly about 5% of the lower flange was exposed with rather deep-seated cracks running a foot or so in both directions from the stripped portion. The remainder of the protection was deeply scored except for about 18 in. at the front of the house where the water could not strike it. There it had retained its original sharp edges. The maximum deflection observed at the center of the slab at end of fire was 3/8 in.

TERRA COTTA—About 80% of one beam protection, including many of the bottoms of the skew-backs, 10% of the other beam protection and the lower faces of six arch blocks, were knocked off during fire and water application (Fig. 6).

At completion of the load test the under surface of the arch was examined carefully. It was found that well over 75% of the lower faces were unsound, portions of which could be easily removed by hand. Practically the entire load was being carried by the central webs and upper faces of the blocks, many of the lower faces being entirely loose, but keyed in by the remaining mortar joints.

This arch had attained a maximum deflection at the end of the fire of 1 1/16 in.

GYPSUM—It will be noted on examination of Fig. 6 that the protection to the reinforcement and to the supporting I-beam flanges in this bay was entirely removed where the water had full play. Toward the front of the house, where this condition did not obtain, the gypsum covering remained.

FIG. 5. BEFORE TEST

FIG. 6. AFTER TEST

FIGS. 5 AND 6. THE TERRA-COTTA (IN FOREGROUND) AND GYPSUM (IN BACKGROUND) SLABS BEFORE AND AFTER FIRE AND LOAD TEST

[Right-hand half of terra-cotta slab plastered on bottom before test.]

The material above the wires was softened, deeply scored and presented a discolored smoky appearance. Four hours of fire produced a total deflection of only 1/16 in.

OBSERVATIONS

CONDUCTED HEAT—Thermo-couples were inserted in the concrete and gypsum arches, from the top, through the cinder fill and into the slab to within 1½ in. of the bottom. Readings were taken from time to time throughout the fire with the result that after four hours' duration the maximum readings were as follows:

Cinder concrete = 620° F. Gypsum = 229° F.

EFFECT OF COOLING—Both the cinder and terra cotta arches recovered after cooling, where as the bypsum settled somewhat. (See deflection table).

EFFECT OF LOAD—The cinder slabs withstood the load of 600 lb. per sq.ft. with slightly over ¼-in. deflection. A deflection of less than ⅜ in. was noted in loading the terra-cotta arch to this amount. After release of load the camber of the arch could be plainly seen by sighting along the lower flanges. When the load upon the gypsum slabs had been increased to 450 lb. per sq.in., the total deflection was ⅝ in. At 496 lb. per sq.in. failure occurred and the slab was propped from beneath to prevent its total destruction.

The Fire on the Harriman Bridge, Portland, Ore.

By John W. Cunningham*

The accompanying half-tones are from photographs taken of the Harriman Bridge at Portland, Ore., after the recent fire. (See news note, p. 323, Engineering News, Aug. 14, 1913.)

Fig. 1 shows the top lateral bracing in the end panel of the fixed span adjacent to the Waddell & Harrington lift span. The laced member at the left is an extension of the top chord (U_0—U_1) which takes no dead load, but receives traction from the lift span. This member as well as the stringers and bracing were buckled by the expansion under heat of the lift span.

Fig. 2 is a view looking toward the lift span, which is raised a few feet above the fixed span in the foreground. The buckled member in the right foreground the inclined back leg of the lift tower. Fig. 3 is a closer view of the buckled tower leg. The flame which affected this member apparently came through an opening of about one square foot burned through the floor near the base of the column.

Figs. 1 to 3. Effect on Structural-Steel Members of Harriman Bridge by Fire in Creosoted Floor System

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For the Sanitary Protection of Drilled Wells to increase the water-supply of Athens, Ohio, the State Board of Health, in approving preliminary plans, requires the city council to prohibit leaching cesspools or other sources of pollution within 500 ft. of any of the wells. Final plans for the additional supply must be filed by Sept 1, 1913. The wells will be located near the Hocking River.

*Northwest Steel Co., Portland, Ore.

The Application of Ultra-Violet Rays for Purifying Large Quantities of Water*

By Max von Recklinghausen†

The industrial methods for purifying water on a large scale have to date been either *mechanical*, viz., settling tanks or filters, or *chemical*, e.g., the adding of sulphate of alumina, chloride of lime, ozone, etc., usually combined with the filtration method.

A new industry has sprung up within the last few years in Europe, dealing with the problem in a new way, which is exclusively *physical*; viz., the purification by ultra-violet rays, previously discussed in the Engineering News.*

This industry is developed to such a point that we think it interesting to describe the state of the art and the principle on which the modern apparatus is constructed.

The bactericidal power of ultra-violet rays is far greater than is usually credited. Exposing clear, but infected water for a small fraction of a second within an inch of the source of the ultra-violet rays, will be sufficient for sterilization.

*The sterilization of water by ultra-violet rays has been discussed in "Engineering News," in the following articles and issues: "Sterilization of Polluted Water by Ultra-Violet Rays" (Sept. 15, 1910); "Sterilization of Polluted Water by Means of Ultra-Violet Rays, at Marseilles, France" (Dec. 8, 1910); Editorial Comment (Dec. 7, 1911); "Water Purification by Ozone, Ultra-Violet Rays and Chlorine Compounds" (Abstract, Oct. 17, 1912); News Notes (Aug. 24, 1911, and Dec. 19, 1912.)

†Managing Director, Westinghouse Cooper Hewitt Co., Ltd., Paris, France.

With this indication in hand, it is necessary only to construct the apparatus so that it will be economically applicable to commercial sterilization.

As a light-giving element, the quartz mercury vapor lamp was chosen, not only on account of its economical production of ultra-violet rays, but also for the fact that the manufacture of these lamps—for the illuminating trade—was standardized some years ago.

Generally speaking, we find the hotter such a lamp runs (i.e., the more energy absorbed by a given lamp) the higher will be its production of ultra-violet rays, limited only by the corrosion of the quartz which begins to take place above 1200° C. The necessity of having the light source at a high temperature forces the adoption as one of the principles for the construction of sterilizing apparatus, to avoid undue cooling of the lamp by the water to be sterilized.

A further principle is to so arrange the apparatus and lamp, that the maximum amount of the light enters the water.

Furthermore, the system must be so applied that the microbes in the water are struck by the light-waves. This means that before the water is exposed, not only large floating bodies must be removed therefrom, but it must also be freed of small visible suspended matter, otherwise germs obscured by such matter will be protected from the devastating effect of the rays. It must be understood that, for ultra-violet rays, the power of penetrating into opaque matters is, if anything, less than for ordinary light.

Rarely do rivers or large springs contain water free from visible suspended matter. Therefore, in most cases it will be necessary to first filter such water for the purpose of eliminating these impurities. Ordinary sand filters will perform this function well, and at a much higher speed than formerly for the production of potable water. An ordinary sand filter may be run at from three to ten times the speed once considered necessary, and still produce thereby physically satisfactory water for the treatment by the ultra-violet rays. Therefore, a smaller capacity filter may be utilized than would be required were it depended upon alone.

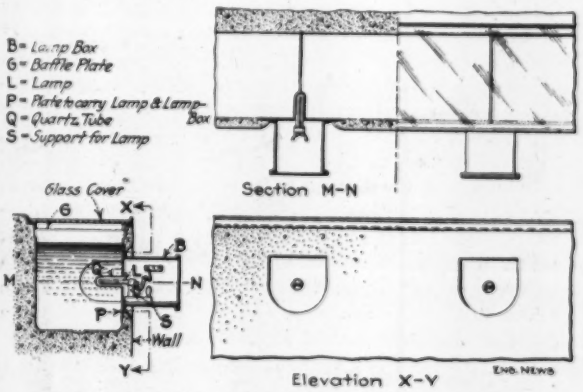
However, as in some cases such filtration will leave some microscopic matter in suspension in the water, we must make sure that such particles receive light on all sides during their passage through the sterilizing chamber. This leads to a further principle in the construction of the apparatus, viz., to bring the water near the light several times, and to create a circulation by means of baffles (built into the sterilizing tank) during its exposure. The small particles will thus be turned over and over, allowing the light to come in contact with all the microbes attached thereto.

The speed of flow past the lamps depends, now, merely on the specific transparency of the water to ultra-violet light, which can readily be determined with sufficient accuracy.

The above apparatus has since been utilized for several public water supplies; one installation (in Marommes les Rouen, a suburb of Rouen, France) has operated nearly three years. In this instance sterilization takes place after the water has been filtered through sand at the rate of 50 ft. in 24 hours. The apparatus sterilizes the water at a rate of 100,000 gal. per day by means of a single 1-hp. lamp. In this case, as in many other applications of this apparatus, the results have always been highly satisfactory whenever properly clarified water passed through the sterilizing chamber, and bacteriologically much purer water was obtained than the best sand filtration plant could have produced.

These results have created a demand for a much larger unit, constructed according to the figure. It consists of the sterilizing chamber or canal into the walls of which are inserted one or several (up to 12) lamp equipments. The lamp equipments consist mainly of the lamp and its support, the lamp box and the quartz protecting tube, which forms a tubular window for the lamp for keeping it clear of the water. Nearly the whole of the quartz tube is filled with the luminous part of a new type of quartz lamp developed especially for this type of apparatus.

The lamps (termed "pistol lamps" from their shape) require 3 amps. at 500 volts direct current. The ultra-violet production of these lamps is ten times as great as for the above mentioned 1-hp. lamp. The U-shaped luminous tube of these lamps permits the entire light-giving part of the lamp to be inserted into the quartz tube. The outer lamp box shown in the figure serves, during the starting of the lamp, as a protection to the operator against the rays.



A Multiple Unit Ultra-Violet Ray Water Sterilizer

One of these canal units, with twelve 2-hp. pistol lamps is sterilizing the water-supply of a fairly large town. The water is obtained from a river, filtered over sand and then passed through one of these units at the rate of 1,700,000 gallons per day. A large part of the time, only a few lamps are in operation. Only when water is fairly dark are all the lamps lighted.

Each unit may be equipped with an automatic electric valve, which prevents the water from getting into the distributing main should the current of all or some of the lamps fail. For large city plants several such canal units should be used in parallel.

The high efficiency and the simplicity of the apparatus, its simple maintenance and small space requirements, seem to open a wide field to its application. The certainty that neither the temperature nor the taste of the water are changed, and that there is no oxidizing action produced which might attack the iron or reinforced concrete of the apparatus, pumps or water mains, are also factors favoring the application of the ultra-violet system for the purification of water.

A Large City Freight Terminal

SYNOPSIS—The new freight terminal for the "Soo Line" at Chicago has as its distinguishing feature the arrangement of all track space, team yards and freight houses on an upper floor or deck, and the utilization of the space at ground level for freight storage. The track deck is a reinforced-concrete slab with an area of about 630,000 sq.ft. (covering several city blocks and streets), supported by some 2000 columns. The deck is covered with ballast, so that tracks can be arranged and re-arranged to suit any changes or developments in the terminal service. The accompanying article describes the general layout and the structural feature of the terminal.

A city freight terminal of exceptional size and presenting some interesting features of design is now under construction in Chicago for the Minneapolis, St. Paul & Sault Ste. Marie Ry. (Canadian Pacific Ry. System), the work being done under the Central Terminal Ry. Co. as a subsidiary concern. The terminal covers eleven city blocks, has its entire area elevated above the street level, and has all the ground area (except cross streets) utilized for storage, while its upper deck is an enormous concrete slab covered with ballast and tracks.

The main portion has its northern end at 12th St. (at which end the buildings are located), and extends some 2000 ft. south to 15th Place, covering the entire area between Canal and Clinton Sts., a width of 325 ft. In this distance it crosses eight streets. Five of these are left open, and pass through the lower level of the terminal. At the southern end there is an extension westward (crossing three streets) to Halsted St., at which point there is a turnout connecting with the present tracks of the Baltimore & Ohio R.R. (Chicago Terminal Line) along 15th St. The plan, Fig. 1, shows the general arrangement and the location of the tracks and buildings. Fig. 2 is a view of the work (July 11), looking south from 12th St.

Description of Terminal

This freight terminal is to serve as the central receiving and distributing station for the "Soo Line," and requires a very large storage capacity. The city would not permit the building of a terminal of this size at the street level, or indeed with any grade crossings. An elevated structure was necessary, therefore, and the problem of arranging the facilities admitted of solution in two alternative ways.

In the first place, retaining walls could be built along the streets and the inclosed areas filled (as on track elevation work), bridges being built across the streets and large storage houses provided in addition to the freight houses required for the actual handling of the freight to and from the cars. This project would have involved 2½ miles of retaining wall 29 ft. high, retaining over 600,000 cu.yd. of fill. To provide adequate storage capacity equivalent to that of the adopted plans would have required an additional warehouse having 520,000 sq.ft. of floor space. If this was built on the same site, over the tracks, it would, of course, be necessary to elevate and lower all the merchandise stored in these upper floors. And the presence of this building would result in the loss of the flexibility in track arrangement which is a special and valuable feature of the adopted design.

The alternative method was to utilize the space beneath the elevated track floor, and this method is embodied in the present plans. The terminal tracks under this arrangement are all carried on a deck structure extending without interruption across the property and the streets, so that the entire area of the property beneath (except that occupied by the

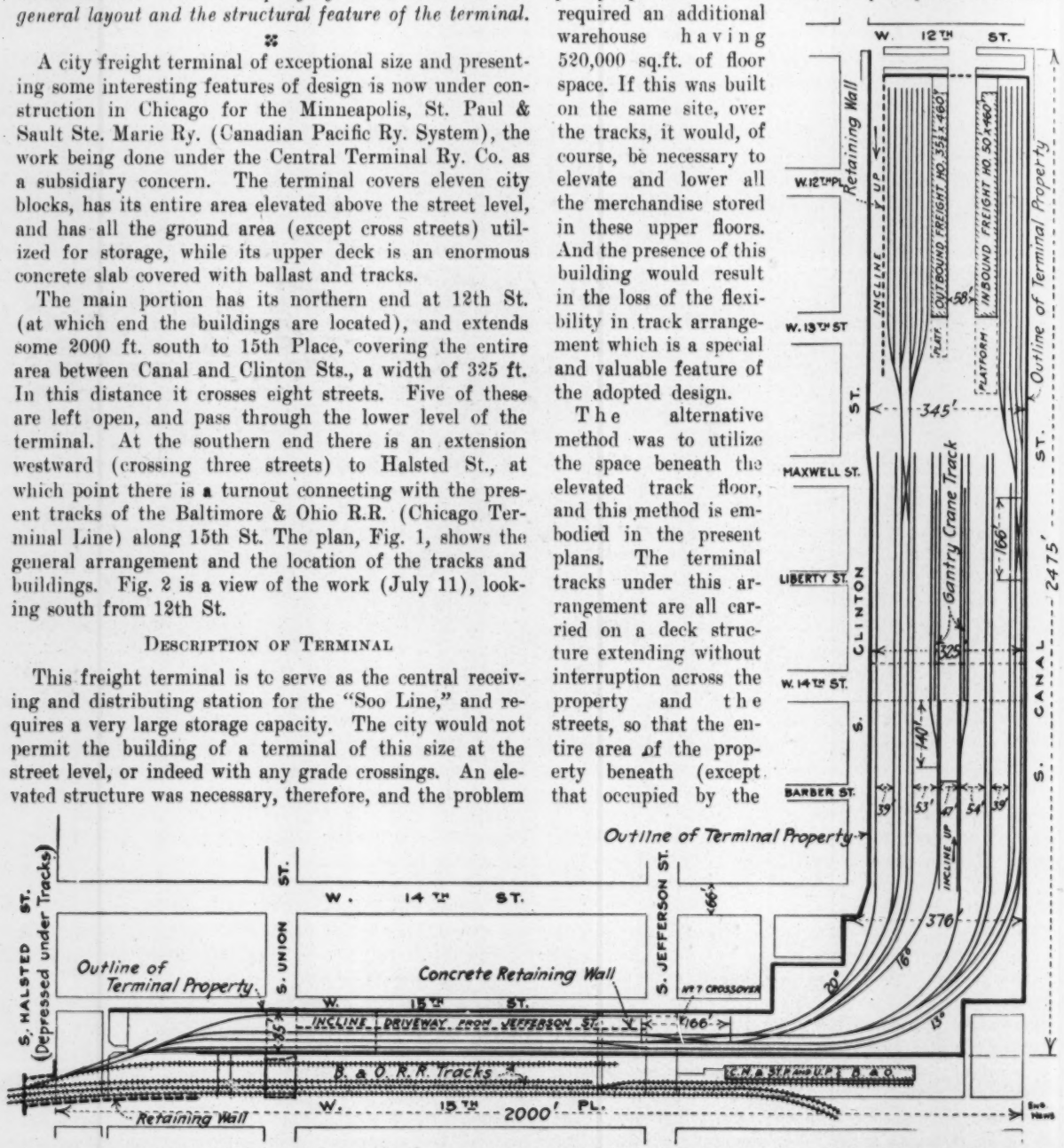


FIG. 1. PLAN OF THE CHICAGO FREIGHT TERMINAL OF THE CENTRAL TERMINAL RY. (MINNEAPOLIS, ST. PAUL & SAULT STE. MARIE RY.)

streets left open for traffic) is available for storage purposes. This available area amounts in all to over 600,000 sq.ft. at the street level, directly accessible to teams for city distribution. From the street the track deck is reached by two inclined driveways, one leading south from 12th St. and the other leading north from 14th Place. These lead directly to the team tracks, the traveling gantry crane (for handling heavy articles), and the driveway between the freight houses.

The Chicago Tunnel Co. is providing several connections with its 60-mile system of freight-carrying tunnels which serve all the principal buildings in the business district by a network of narrow-gage electric railway. The tunnel company will have a station at the street level at 12th and Canal Sts., under the inbound freight tracks. Three elevators will connect the tunnel tracks with the tracks of this freight station and with the tracks in the freight houses of the Central Terminal Ry. above.

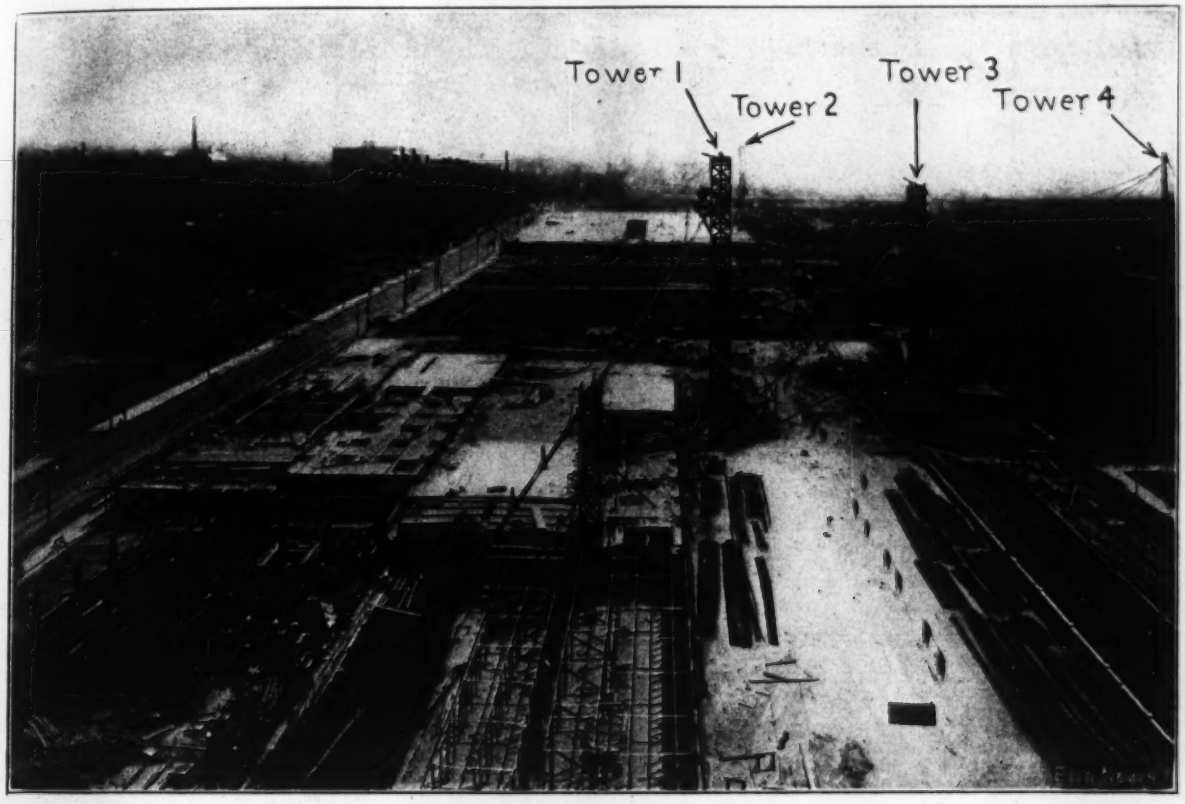


FIG. 2. GENERAL VIEW OF TERMINAL UNDER CONSTRUCTION, SHOWING PART OF THE CONCRETE DECK OR TRACK FLOOR WHICH WILL COVER ELEVEN CITY BLOCKS

(At the right are some of the steel reinforcing units for the columns.)

BUILDINGS AND TRACK LAYOUT

The freight houses and office building are located at the north end of the terminal, adjacent to 12th St. The main inbound and outbound freight houses are to be built at present with a length of 500 ft., but provision is made for extensions of 300 ft. While the buildings will be structurally of concrete, they will have a veneer or facing of brick with cast-concrete trimmings.

The inbound freight house is to be built four stories high but is designed for two additional stories when increased space is required. The building will be served by five tracks along the Canal St. side, with a capacity of 80 cars. The floor space is about 100,000 sq.ft., and will be double this amount when the extensions are made. The plans call for a cold storage chamber on the first floor.

Five elevators will distribute the freight received at the second floor to the upper and lower floors. One elevator from the freight tunnel will serve the second floor directly. Three spiral-chute conveyors will be installed. Also one scale on the first floor and four on the second floor, all set flush with the floor at the shipping doors. The plans provide for 14 team shipping doors on the first floor and 19 on the second. The entire east side of the building will be open, with columns 20 ft. c. to c. and with doors of full width between the columns, so as to permit cars to be spotted with great ease. A screen wall of concrete and brick will conceal the freight tracks from view from 12th St. Boulevard.

The outbound freight house is to be two stories in height, and served by eight tracks with a capacity of 105 cars. The present floor area will be 65,000 sq.ft., with provision for future increase. The freight will be received from the driveways through 16-team doors on the first floor and 18 on the second. These doors will be served by 15 scales, set flush with the floor. The entire west side of the second story will be open to the tracks, with columns 20 ft. c. to c., and doors of full width between them. Two freight elevators will serve the first and second floors. A 10-ton pillar crane of 20-ft. radius for handling heavy castings, etc., will be located on the

outbound freight platform, so as to serve cars, trucks, and platform from one fixed position.

Between the two freight houses will be a 58-ft. driveway on both the ground and second floors. The ground floor driveway is entered directly from 12th, Maxwell and Canal Sts. The other is reached by the inclines from 12th St. and 14th Place. The floors of the freight houses are 3½ ft. above top of paving. The second floor of the inbound house is 4 ft. above top of track rail, while that of the outbound house is 4 ft. 3 in. above top of rail.

Structural Design

The structure is designed to carry any desired arrangement of tracks at 12-ft. centers and under the heaviest loading which it is considered may be imposed upon them. The track slab or deck is designed for the E-50 loading of Cooper's specification. This deck is absolutely unbroken by beams and is to be ballasted, permitting of unlimited elasticity in the track arrangement. Fig. 3 is a cross-section of the structure on the center

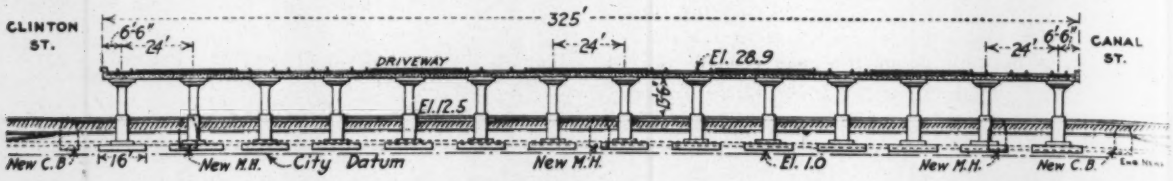


Fig. 3. Cross-Section of Terminal, with Tracks and Team Driveways on the Upper Deck and Storage Space at Ground Level

The team tracks are located south of Maxwell St. and have a total capacity of 260 cars, which can be greatly increased when the need arises. The flat, unobstructed deck permits the tracks to be re-arranged at any time as experience may show to be desirable. A traveling gantry crane will serve four tracks near the south incline driveway; and a scale 9x20 ft. for automobiles will be located conveniently to the inclines and driveways.

Office Building and Power Plant

The freight and business offices of the M., St. P. & S. S. M. Ry. will be located in a building 143½x36 ft., fronting on 12th St., at the north end of the freight houses. This will be four stories high, to be increased later to six stories. The ground-floor driveway entrance is through this building.

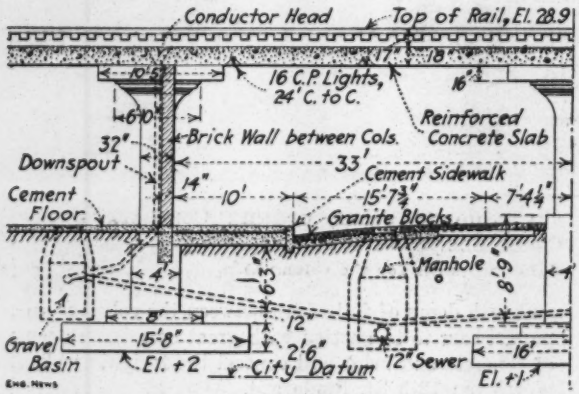


Fig. 4. Section of the 14th St. Subway

(The column spacing is 33x24 ft. in the subways. In the blocks it is 24x24 ft.)

The power and heating plant is located under the outbound tracks, near 12th St. The coal will be received on the west track and dumped directly into the bin below; the ashes will be hoisted to cars on the same track. Ultimately all heat and power required for the operation of the terminal will be developed by this plant, and provision has been made for its enlargement. The boilers are served by a reinforced-concrete chimney 140 ft. high.

line of 14th St., Fig. 4 is an enlarged detail of this 14th St. subway. Fig. 5 is a view of a portion of the area, showing the reinforceing bars in place and the method of concreting, which is described below. Fig. 6 is an interior view, showing the columns and heads. Figs. 2 and 7 give an idea of the vast size of the concrete slab.

The structure is built of reinforced-concrete throughout, with the exception of the columns at the track side of the inbound freight house. Concrete was chosen because of lower first cost, lower maintenance cost and greater permanency than structural steel. The use of steel would have required through girders at the street crossings (destroying the elasticity of the track arrangement), or would have necessitated raising the entire structure several feet higher above the street. It was considered that structural-steel construction of the same strength, and properly fireproofed, would have cost very much more than the present concrete structure.

Various types of reinforced-concrete construction are to be found in the structure, the requirements of each case being studied and the type best suited being used.

For the track deck, with the heavy load of ballast and trains, calculations showed that the flat-slab construction with depressed heads led all other types by a very wide margin, from the standpoint of cost and durability in every way. This construction also gave a minimum of thickness over the streets, which in turn determined the elevation of the entire structure. It is the opinion of the designers, in the light of their computations, that for this sort of structure the flat slab is without a rival.

In the freight houses also, with a loading of 350 lb. per sq.ft., the flat-slab design was shown to have considerable advantage in cost over its nearest competitor, as well as making possible a more attractive building. For the office building and the outbound freight-house roof, where the loads are light, tile-concrete joist construction was adopted as being cheaper and somewhat superior in regard to condensation and noise conductivity. For the driveway slab between the freight houses a reinforced-concrete girder, beam and slab construction was found most suitable, as the panels were rectangular (about 20x30 ft.) instead of square. The retaining walls are of a semi-gravity type, being lightly reinforced for temperature and bending stresses.

The normal spacing of the columns in the blocks between streets is 24 ft. c. to c. in both directions, except that beneath the buildings the longitudinal spacing is 20 ft. These columns are circular, with curved or trumpet-shaped heads and rectangular caps in which the head reinforcement is imbedded. At the streets the span is increased to 33 ft., with one row of columns on the property line and one in the middle of the street (Fig. 4). These columns are of square section, tapering to a deep-flaring cap also of rectangular section. The parapet wall is increased in depth over these long spans, as a girder. Where the length of block between streets does not divide into 24-ft. spaces, the necessary variation in length of span is made in the panels next to the street or subway spans.

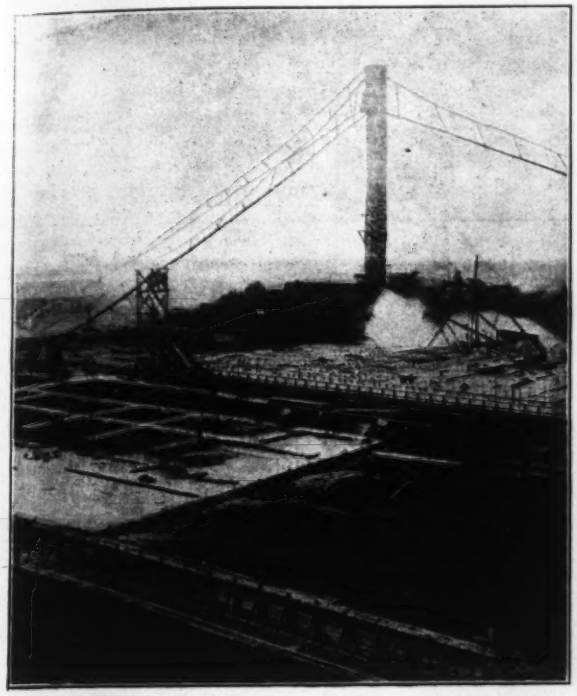


FIG. 5. CONCRETING THE 18-IN. REINFORCED-CONCRETE SLAB FOR THE TRACK FLOOR

The thickness of the slab is normally 18 in., with an increased amount of steel reinforcement for the 33-ft. street spans. There are no expansion joints. The clear headway over the streets, as required by the city ordinances, is 13 ft. 6 in. at streets having car tracks and 12 ft. at other streets.

At the driveways, where the beam and girder construction is used, the column spacing is 30 ft. east and west and 20 ft. north and south. Here there are three parallel (longitudinal) girders 30 ft. apart. Between the girders are cross-beams of the same depth, about 6.6 ft. apart, each 20-ft. spacing of the columns having beams over the columns and at two intermediate points.

On account of the vibratory nature of the load, deformed bars were used very largely. Some square-twisted bars were used and a small amount of plain round bars; also large square bars in the girder construction.

The foundations are simple, as the entire area is underlaid by a deep bed of the stiff blue Chicago clay. The column foundations are rectangular footings, reinforced in the bottom course, which rests on the clay at about 2 ft. above datum (Chicago River level). The top of the slab is at an elevation of 31.18 at Jefferson St., and slopes down to the north at a very slight grade to 26.23 at the freight houses.

The appearance of the vast covered space with its rows of supporting columns (Fig. 6) is very striking, and although there is very little attempt at decoration, the effect of the work both in general and in detail is very pleasing. While the buildings will be structurally of concrete, they will have a veneer of brick and cast-concrete.

WATERPROOFING

As noted above there are no expansion joints in this immense slab, but a small amount of reinforcement is placed in the surface of each panel to resist temperature stresses. The entire deck has a slight slope down to the north. It is arranged for drainage to 5-in. downspouts spaced 48 ft. apart in both directions, the surface of the slab being inclined to the central high points between the downspouts, thus slightly increasing the thickness. The concrete itself is very dense, but to provide against any leakage through the concrete or any small cracks, the entire surface of the slab (about 630,000 sq.ft.) will be covered with a three-ply membrane of felt and burlap well mopped with asphalt. In general this membrane will be sealed to the slab only at the downspouts and walls, thus enabling any expansion and contraction movements of the concrete to take place without affecting the membrane.

Upon the membrane will be a 1-in. protective course of asphaltic mastic, the composition of which is prepared with a view to giving ample elasticity in cold weather, as the ballast covering is expected to prevent it from reaching such a temperature as to be liable to flow in hot weather. The mastic is composed of Trinidad asphalt and gilsonite, with torpedo sand, gravel and screenings. The preparation is made in a hot mixer, having a heated revolving drum.

TRACK, BALLAST AND DRIVEWAYS

The entire area of the deck will be covered with stone ballast, having a thickness of 6 in. under the ties. This ballast will be laid on the mastic course noted above. The tracks will have ties and rails laid in the usual way, and the driveways and teamways will be of concrete laid upon the ballast. The purpose of this arrangement is to

FIG. 6. INTERIOR OF STORAGE SPACE ON GROUND FLOOR, WITH COLUMNS SUPPORTING THE TRACK FLOOR

allow ample flexibility for changes in the track plan. Tracks can be taken up and a concrete paving laid in their place on the ballast; or a concrete driveway can be torn up and tracks laid upon the ballast beneath it.

The track layout is shown in Fig. 1. The tracks at the freight houses will be laid 11.75 and 12 ft. c. to c., with No. 6 frogs and 12-ft. switch rails. The turnouts and the diamond or double crossovers will have No. 7 frogs and 15-ft. switch rails, with a length of lead of 63 ft. 3 in. for the crossovers. The ties will be 7x8 in., 8 ft. long, spaced 24 in. c. to c.; switch ties will be 6x8 in., 20 in. c. to c. Cedar ties will be used on straight track and creosoted oak ties on switches and curves. For each switch there will be two headblocks 6x9 in., 14 ft. long, to carry the switchstands. The tracks for the gantry crane will have creosoted oak ties 6x8 in., 23 ft. long, 24 in. c. to c.

Construction Work

The concrete mixture used in the reinforced work is 1:2:4, made sufficiently wet to flow freely through the network of steel bars. The bars near the surface of the slab (to reinforce it against temperature stresses) are placed as the concrete is deposited. The bars in the column heads are held at proper elevation by small concrete blocks. The bars of the bottom layers of the slab panels are kept at a sufficient height from the slab form by small sheet-metal supports, special supporting clips being used at the intersections of bars. For the plain or non-reinforced concrete a 1:2½:5 mixture is used; but there is a relatively small amount of this, mainly in the ground floor and the driveway base. All concrete is made with broken stone.

All the reinforcing steel in the track slab is bent on the work, which results in a large saving, and in better work. The ¾-in. round bars used on the larger portion of the work are found to bend very easily. The 1-in. twisted bars used in one section are very much more difficult to bend, but even with these it is believed that bending in position is cheaper than and preferable to shop bending.

The great amount of concrete and the necessity of carrying on the work rapidly and continuously makes it necessary to keep a large supply of material on hand. This was one of the troublesome problems in the construction work. There were three concrete-mixing plants, and at only one of these was there much storage space available. Probably 90% of the stone and sand was handled directly from cars to the bins serving the mixer. This necessitated holding about 36 cars of material on hand, and at times led to considerable expense for demurrage. The materials were unloaded from cars on spur tracks by means of derricks and locomotive cranes operating grab buckets, which delivered the materials either into the stock pile or into the elevated bins from which they are delivered to the mixer by gravity. At two of the plants the stone and sand bins were of 200 and 100 cu.yd. capacity, respectively; at the third plant they were of 100 and 60 cu.yd. capacity. In view of the liability of interruption to the supply of cement, owing to the usual shortage of cars in the fall, a large amount of cement in sacks was stored in the space under the completed deck, to be used only in case of such emergency.

Each mixer plant comprised the storage bins, a drum mixer and an elevator tower, most of the concrete being

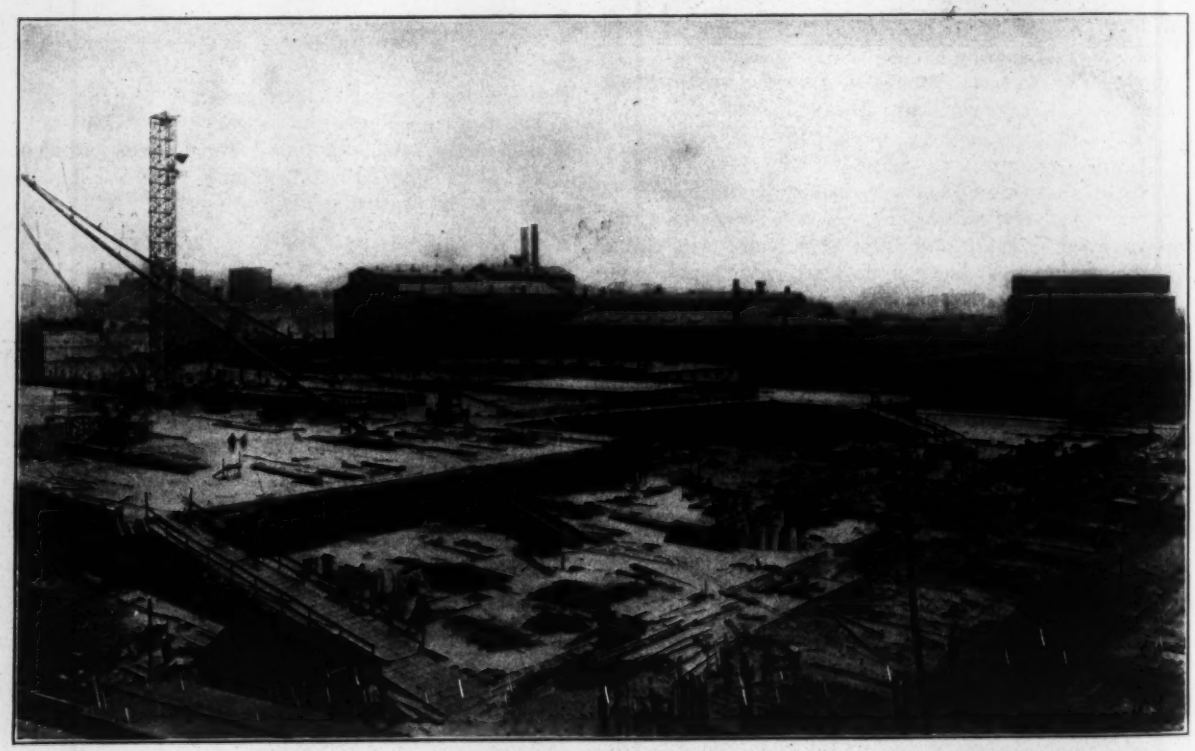


Fig. 7. Construction Work on the Chicago Freight Terminal for the M., St. P. & S. S. M. Ry.
(This view shows various stages of the work, from excavating the column foundations to concreting the upper deck or track floor.)

distributed by inclined chutes or open troughs leading from these towers. The two larger mixer plants had each a mixer of 33-cu.ft. capacity, and an elevator bucket of 10-cu.ft. capacity; one served a 150-ft. tower and the other served a 125-ft. main tower and also a 100-ft. intermediate tower. The third and smallest plant had a 22-ft. mixer and a 20-cu.ft. bucket, with a tower 100 ft. high. Two additional mixers are erected for use in case of any accident to one of the regular machines at a time when it would be undesirable to stop concreting. Such a case did occur, when a mixer broke down just when a section of work was being finished. Hand mixing had to be employed in this emergency, and as a result these reserve machines were installed. They will be used later for mixing concrete for the paving and other auxiliary work. Fig. 2 shows the four main towers, with an auxiliary tower at the right.

The concrete distributing chutes have an average inclination of 4 to 5 in. per foot, and are suspended at intervals by cable slings from cables which run from the top of the tower to anchorages in the ground, as shown. Near the head of the tower the elevator bucket dumps the concrete into a hopper, from which it is discharged into one or other of the chutes, the distribution being regulated by a man at the hopper. Sometimes it is discharged simultaneously into two or even three chutes, with satisfactory results. At some points, a hopper on an intermediate tower intercepts the flow in the main chute and diverts it to a second chute in another direction.

Fig. 7 shows several stages of the work. At the right are some of the pits for the column footings; also the cylindrical forms for columns, the falsework and piles of bars. At the right is a part of the completed deck on its columns, with two temporary driveway inclines. One of the towers and several chutes of the concreting plant are shown, one of the chutes ending in a hopper from which the wheeled tubs are filled.

The chutes end in hoppers fitted with slide doors in the sides, Fig. 8, through which the concrete is discharged into wheeled tubs or carts of 6-cu.ft. capacity, for delivery to the forms. A small amount of narrow-gage track with steel dump cars is on hand, and has been used to a limited extent where the length of haul was considerable. This equipment may be used for distributing the stone ballast for the tracks.

In concreting, the aim is to cover large areas at once, including the columns and the slab deck. At street crossings, an area of five panels in length and three panels in width is usually taken, so that there is absolute continuity of the panels forming the bridge. The forms for the columns and column heads are of sheet metal. All other form work is of wood, and a forest of timbering is required to support the great area of floor forms for the concrete deck. There are some 3,000,000 ft. B.M. of lumber in the forms and supports, and the lumber is used three and four times.

The output of the larger plants is from 450 to 480 cu.yd. per 8-hr. day. One plant has a record of 608 cu. yd. in 10 hr., averaging an elevator bucket of concrete every 50 sec. The smaller plant has an output of 350 to 400 cu.yd. per 8-hr. day. It will be evident that on so extensive a piece of work a very large force of men is required. The number averages 1000 to 1200, but at times has been as high as 1400.

Progress of Work

Construction work was started on the footings in August, 1912, and continued with very few interruptions throughout the fall and winter. The force was increased as rapidly as the clearing of the site and acquisition of property would permit. There are about 2000 columns in the project and by June 15, 1913, all but about 75 of the 1370 footings had been completed. In May, the construction of the track slab was begun and by the above date three city blocks had been completed, together with the bridge across Union St. and the slab across 14th Place. Work on the freight houses was delayed owing to difficulty and delay in acquiring certain property, but practically all the interfering buildings have been razed. The work is now being prosecuted at high speed and it is expected that the terminal will be completed by January, 1914.

Some of the quantities involved were originally estimated as follows, although the final figures may be slightly different:

Excavation	125,000	cu.yd.
Concrete: track slab and columns	36,300	cu.yd.
Footings	25,300	cu.yd.
Freight and office buildings	13,000	cu.yd.
Retaining walls	13,350	cu.yd.
Driveway and pavements	17,700	cu.yd.
Total, about	108,000	cu.yd.
Reinforcing steel	6,500	tons
Cement	180,000	bbl.
Area of ground covered	18.5 acres	(800,000 sq.ft.)
Area of working space	34.7 acres	(1,500,000 sq.ft.)
Area of working space under roof	17.9 acres	(780,000 sq.ft.)

Arrangements are being made to measure the actual stresses in the steel and concrete of the track slab. For this test, four adjacent panels will be loaded with locomotives, and complete measurements made of the resulting stresses under various critical loadings. Dead-load stresses occurring when the forms are removed will also be measured.

Engineers and Contractors

This extensive freight terminal of the Central Terminal Ry. was planned under the direction of C. N. Kalk, chief engineer of the Minneapolis, St. Paul & Sault Ste. Marie Ry. The engineers and contractors are the Leonard Construction Co., of Chicago, with H. H. Hadsall in charge of the design and construction, and W. M. Kallasch as superintendent on the work. The architect is H. Hanselmann, of Chicago. The structural design was licensed and checked by the Concrete-Steel Products Co., of Chicago. The design and cost computations were made by Arthur R. Lord, as consulting engineer of the Leonard Construction Co., and the design of the track slab was submitted to and approved by Prof. W. K. Hatt, of Purdue University. For plans, description and other information we are indebted to Mr. Kalk and to the Leonard Construction Co.

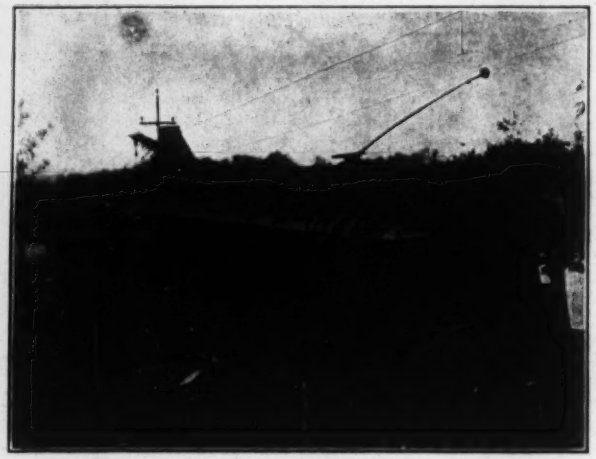
The Manufactures are the Chief Feature in the Growth of the Export Trade of the United States, according to the Department of Commerce, Washington, D. C. They have increased during the ten years ending 1913, 153% (exclusive of food-stuffs). The highest record was made in the fiscal year 1913 when imports also rose to their highest value in the same.

Receipts from the National Forests of the United States were nearly $2,500,000 for the year ending June 30, 1913. About half of the receipts were for timber. During the year the Government let contracts totaling $4,000,000 for the sale of timber to be cut at once or in the future. Of the gross forests' receipts, 35% go to the states in which the forests are located, to be used for schools and roads.

A Butting Collision near Oswego, N. Y.

A butting collision involving two cars on the Empire United Ry. and injury to 35 persons, occurred two miles south of Oswego, N. Y., Aug. 6. The accompanying photograph shows the condition of the cars after the collision.

The facts of the accident, according to the operating manager of the company, seem to indicate that the northbound car was proceeding down a 5% grade at about half speed when the motorman sighted the oncoming southbound car. He is claimed to have promptly applied the air-brakes and brought his car almost to a stop



HEAVY INTERURBAN CARS TELESCOPED IN COLLISION

at the moment of collision. The northbound car, being on the down grade, mounted the other and telescoped it for a distance of 24 ft., indicating that the latter was proceeding at a considerable rate of speed although the actual speed has not yet been established.

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U. S. Geological Survey Topographic Maps for the Civil and Mining Engineer*

BY E. G. WOODRUFF†

Few authors of treatises and papers on engineering subjects have given adequate attention to topographic maps. The statement applies especially to mining engineering in all branches. Even those who have discussed such maps have treated the subject only in a general way. Therefore it is proposed to outline in this paper a few of the uncommon yet important relationships which topographic maps and their byproduct, structure-contour maps, have to modern methods of mining investigation and development when the facts depicted on such maps are properly interpreted.

To a large extent contour topographic maps are the result of the demands of engineers. Since the maps have been made chiefly for the use of the engineer, they ought to meet his demands on the one hand, and, on the other, should be used by him to the fullest extent. Such maps, when properly made, accurately portray a portion of the earth's surface and should present the topography better than a personal examination of the area without a map could present it, because the person making such examination views only a limited portion of the surface at a time, and estimates distances only roughly with his eye.

*Abstract of a paper read at the Butte, Mont., convention of the American Institute of Mining Engineers, Aug. 18-21; printed in full in the June "Bulletin" of the society.

†Geologist, U. S. Geological Survey, Washington, D. C.

Let the engineer consider carefully the fact that in many instances the topographic map is superior to a personal examination of the surface features of a property because the map is based on careful instrumental measurements, whereas the eye gives imperfect data because it can be used merely for the estimation and not for the measurement of distance. Such estimation is frequently unreliable, since it depends upon the engineer's physical condition.

When weary in the evening, after a hard day's tramp, he views a cliff or steep slope which he may have to climb in an entirely different way than he views it when he is fresh in the morning. The long mountain slope appears different in the ascent than during the descent. It is not uncommon for the traveler approaching the steep mountain front from the plains to feel that he is gradually descending, whereas he is actually going up a gentle slope. The experience is so common that, in irrigated regions along the mountains, the feeling is expressed in the conclusion that water runs up hill. These familiar experiences will no doubt suffice to demonstrate the statement just made, that, in some ways at least, topographic maps give better information than field examination.

More than half a million topographic maps of the U. S. Geological Survey are now purchased annually for work in the mapped portion of the United States, which is 38% of the total area. At this same rate, if all the country were mapped, the demand during the year would have been for 1,400,000 maps. Furthermore, it seems very conservative to consider that at least half of the maps purchased during the preceding year would have been used also. On this assumption—namely, that the additional maps might have been sold, and that half of those sold during the preceding year were used—we infer a potential demand for the use of 2,000,000 topographic maps.

Both surface and subsurface features are now represented by contour maps, the former by the topographic maps and the latter by the somewhat less known structure-contour maps. Surface work is directly related to topographic maps and only indirectly related to structure-contour maps, whereas with underground work the reverse is true.

Therefore, in this paper the two kinds of maps will be treated more or less independently. Furthermore, since most mining engineers are familiar with topographic maps and their uses, this paper discusses only certain data, presented on topographic maps, but generally overlooked by the engineer because he fails either to read the map properly or to use the information which it contains.

One of the common problems in both the development and the operation of a mining property is to control the water supply which is to be used, and to protect the property from any excess that is not needed. In many plants the flood water of a nearby stream is a menace. It takes years of measurements to determine the high-water stage of a stream, but a close approximation may be made in another way.

With the size of the drainage basin given and the daily rainfall known, the stream's flood stage can be roughly estimated. In the United States the observations of the Weather Bureau have given us data on the rainfall for most parts of the country. The other factors, such as the size and characteristics of the drainage basin, are shown on topographic maps. In this way two of the important elements in the solution of the water problem can be obtained.

But more data can be actually obtained by proper interpretation of the map, especially with regard to flood. If the slopes are steep the run-off occurs rapidly and the streams are quickly flooded, but if the slopes are gentle the flow of the water is more gradual and lower flood stages may be expected. These considerations are not theoretical, but actual.

Of course, the runoff as a result of the rainfall must pass under the bridges and culverts of a railroad. These bridges must be of a certain size in order to permit the water to flow beneath, so as to avoid washing out the grade. Engineers recognize that, with their general knowledge of conditions and with specific data on the amount of rainfall and on the size and topography of the basin to be drained, they can closely estimate the maximum amount of water which may be expected to flow under any given bridge and design the structure accordingly.

A knowledge of the topography of a drainage basin is applicable in another way. Snow slides have wiped out more than one mining camp. With the fragrance of the pine enveloping him in the balmy days of summer, when claims are usually examined, the engineer does not fully appreciate the dangers from a winter's accumulation of ice and snow above the property in question. Possibly, as stated above, his point of view cannot give him the proper relations of slopes, which might warn him of the danger.

Yet many of us have seen those streaks on the mountain

side bared of their normal verdant cover by the avalanche and know that slides are a menace. We have seen too that, as the avalanche plunged into the narrow valley below, the width of the area devastated was less, but the destructive power increased. Do all geologists and engineers, however, know that the topographic maps present data for the proper location of a camp to prevent its destruction by snow slides? The snow must accumulate somewhere, and it can slide only on steep slopes. The size and slope of the catchment basins and the angle of slopes over which the snow will move are shown on topographic maps. Reliable results can be determined from these data.

Commonly, topographic maps are used as a general guide and accurate surveys are made subsequently as a basis for detailed work; but in many cases the second and more expensive survey is unnecessary, because the accurate location of points can be determined from data shown on topographic maps. In this work the map is used as a base and needed refinements can be added to it.

It may be that the map available is drawn with 50-ft. contours, whereas a 10-ft. interval is required for the work in hand. If so, the map may be accepted and supplemental contours and other refinements may be added. Ordinarily, the geologist takes his topographic map as a good guide, and a standard map is deserving of this confidence, but even good things have their limitations. On a 25-ft. contour map the elevation between the two lines is generally estimated. In this case the error cannot be more than 25 ft. Likewise, horizontal positions are more or less a matter of estimation within certain limits, but the engineer is constantly demanding accurate measurements, not approximations. Good topographic maps like those of the U. S. Geological Survey furnish a basis for this accurate work.

To obtain these precise data it is necessary for the engineer to repeat some of the operations of the topographer at the points where such data are desired. To do this work it has been found convenient to place the topographic sheet on a suitable-sized planetable board, and use it as an original sheet; or, better still, to have a photolithograph printed on good drawing paper on a scale suitable for the work in hand.

The engineer proceeds to the field equipped with his sheets, telescopic alidade, and stadia rod. The benchmarks and triangulation stations shown on the sheet, give him points from which he can establish his locations. This is an important feature. Here are useful data presented on topographic sheets and seldom considered, yet they furnish a basis for accurate work. They comprise some of the best data on the sheet, such as triangulation stations, houses, water towers, oil derricks, etc.

Some points are shown only by benchmarks, and the signals should be reëstablished. If this is done for the engineer when he enters the field, he proceeds to the point whose location and elevation are desired and orients his table by the magnetic or three-point method, locates his point by intersections, and determines his elevation by vertical angles. After he has determined the location and elevation of his station he records the observations about the outcrop of the vein or bed.

So far, the engineer has worked alone and he can continue to do so if necessary, but if he is accompanied by an assistant who can serve as a rodman, or, better still, who can operate the planetable, the engineer, serving as a rodman, traces the outcrop and holds the rod at points to be located.

The planetable operator sights to each stadia station and determines the location and elevation accurately. With these data he can plot the outcrop of a vein, if that is what he is tracing, though it is exposed intermittently, and determine its position with regard to land lines which are also shown on the sheet.

Furthermore, he can obtain data from which he can determine the pitch of the vein. For the sake of illustration, a simple problem is assumed by supposing that the engineer has found three exposures of a vein, two at about equal elevations on opposite sides of a valley, and a third in the bottom of the valley, 500 ft. lower. If the three fall in a line the vein is vertical, but if the valley point falls to either side of the line joining the other two the vein dips in the direction of that point in an amount equal to the distance of the point from a line joining the two points on the sides of the valley. In the case assumed above, if this distance of the point from the line is equivalent to 100 ft. the vein dips 100 ft. in 500, or 20 ft. to the hundred.

The remaining part of this paper relates to structure-contour maps, or maps that might well be called strata-form maps because they portray the form of the strata. Their discussion is introduced because they are very closely related to surface-contour maps in their construction, interpretations and uses. Such maps are in demand in coal fields, and especially in oil and artesian water fields, but are also applicable to some metalliferous areas. The methods of construction of such a map vary with conditions.

These maps show the structure of some distinctive stratum, such as a coal bed, oil sand, or any persistent bed. Contours are drawn on equal elevations of the bed just as surface contours are drawn through points of equal elevation of the surface. Synclines are depressions just as valleys are depressions, and anticlines are ridges in the strata just as mountains and hills are land ridges.

To illustrate the relation of structure contours to topographic contours, let us suppose that we have strata folded into a syncline and an adjacent anticline, and that a definite unbroken stratum is the surface rock over both. Then the structure and surface contours are coincident if the same datum plane is used for both. Sea level is generally used in the construction of both kinds of maps. The structure lines and topographic contour lines agree exactly.

Now, if a portion of the anticline is eroded and the débris partly fills the valley without attendant crustal movement, the structure contours remain the same, but the surface contours are shifted. On the crest of the anticline the surface contours show lower elevations than the structure contours, whereas the reverse is shown in the valleys. In the construction of the map, however, the portion of the structure-contour map representing the crest of the anticline is generally shown by broken lines to indicate that the bed does not actually remain and that its position has been assumed. In the adjustment of the assumed eroded surface the contours are shifted from their former position of coincidence to new ones, depending on the amount of erosion or deposition.

If the syncline and anticline trend north-south, the structure contours extend in the same direction; but if the syncline is filled so that the surface slopes to the north or south, the topographic contours extend east-west across the structure contours. The difference in elevation of any two lines at the point of intersection is the distance of the contoured strata from the surface.

The depth at points between contour lines can be determined by interpolation. Thus, if a point lies midway between two surface contours, 2100 and 2150 elevation, its altitude is assumed to be 2125, and if the same point is two-fifths the distance from the 1150-ft. structure contour to the 1200-ft. contour its elevation is assumed to be 1170, and the depth of the bed below the surface is the difference between 1170 and 2125, or 955 ft.

The uses of such maps are apparent and their accuracy is surprising. One such map, made by M. R. Campbell, of the U. S. Geological Survey, is published in the "Masontown-Uniontown Folio," No. 82. The structure lines are drawn on 50-ft. contour intervals to indicate the position of the Pittsburgh coal west of Laurel ridge, and the Upper Freeport coal east of that ridge. Diamond drilling and mining subsequent to the construction of the map have shown that the maximum error is only a few feet.

Of course, it is unnecessary to point out the value of such a map to the mining engineer in studying a mining problem, or to the civil engineer in studying a foundation problem. Supposing he has both surface and structure-contour maps printed on one plate, as they usually are, then from the one sheet he can select the possible locations for the surface works, can infer the depth to the coal if it is a coal mine, the lowest point for drainage purposes, the slope for haulage, etc.

NOTES FROM THE ENGINEERING SCHOOLS

UNIVERSITY OF WASHINGTON—Two sections of land in the Snoqualmie National Forest has been allotted by the U. S. Department of Agriculture for use in the practical forestry courses of this university.

MASSACHUSETTS INSTITUTE OF TECHNOLOGY—The Stone & Webster Engineering Corporation, of Boston, has been selected as construction engineers for the new plant of the Institute in Cambridge. Work will begin at once on the general preparation and drainage of the site and on building foundations. A little grading has already been done and this will be continued on a larger scale. Finished drawings of the structures will not be

completed until fall, the study of locations and other essentials for the different departments being only recently completed.

A Landslide Buried a Steam Shovel and killed 13 men at the Portobello quarry, Panama, Aug. 11.

A Portion of a Steamboat Dock Collapsed at Saugatuck, Mich., Aug. 6, precipitating 30 persons into 7 ft. of water. No one was drowned.

A Locomotive Split a Switch Frog on the Indianapolis & Vincennes division of the Vandalia R.R., near Centerton, Ind., Aug. 12, and was thrown from the track by the derailing switch. Four trainmen jumped, sustaining slight injuries.

The Snapping of the Cable Pin allowed two ore cars carrying 12 tons of ore and 13 miners, to dash down a 38-deg. grade for a distance of 3300 ft., at the Coronada mine, near Clifton, Ariz., Aug. 14. Three of the men rolled off the cars when the latter started their descent; the other nine were killed.

The Cause of the Explosion and Fire which wrecked the plant of the Husted Milling Co., of Buffalo, N. Y., on June 24 (described in "Eng. News," July 10), and resulted in the death of 33 men, was not determined by the inquest, although about 110 witnesses, mainly employees who were at work on the day of the explosion, were examined. The company, however, was exonerated from the charge of criminal negligence, in a decision handed down by Judge Thomas H. Noonan, in the city court, on Aug. 15, in which he said:

> The testimony without exception has been that the plant on that day (June 24) was in good working order, and that all ordinary precautions for the protection of the men had been taken. No evidence has been produced that would throw any light upon the cause of the accident, and I therefore find that the men whose deaths were caused by the accident, came to their deaths from a fire originating from causes unknown, and that there was no culpable or gross negligence on the part of the Husted Milling Co. or its officers or employees, and there is therefore no ground for the issuance of a warrant for manslaughter against anybody.

American Engineers in Peril in Mexico—A dispatch to the New York "Times" of Aug. 14, states that Wm. B. Fuller, chief engineer and general manager of the Mexican Northern Power Co., which is building the La Boquilla dam on the Conchos River, with headquarters at Santa Rosalia, in Chihuahua, Mex., has just been able to reach El Paso, Tex., after many threats against his life and personal liberty by the Federal general in charge of the territory. Mr. Fuller immediately notified friends, who urged the State Department to look into the safety of Shirley C. Hulse, who, with Mrs. Hulse and a six-year old daughter is at the construction camp of the dam, as principal assistant engineer. In Mr. Fuller's opinion, the Hulses are, with other Americans, in considerable danger.

Mr. Fuller is a well known New York engineer, whose article in "Engineering News," May 23, 1912, page 1002, on "Dam Building and Bullet Dodging," will be remembered. Ever since that time, the construction camp has evidently been the scene of considerable trouble. Mr. Hulse is a young Cornell engineer, who contributed an article recently in "Engineering News," showing a bridge which had been destroyed by the revolutionists.

A later dispatch to the same newspaper states that Mr. Hulse and his family finally reached safety at Chihuahua City. The Mexicans on the river below the dam are afraid the dam will deprive them of their water supply; hence their hostility to the project.

Smoking on New York Street Cars—The Public Service Commission has adopted an order directing all street railroad corporations in the city of New York to prohibit smoking or the carrying of lighted cigars, cigarettes and pipes on the cars or the platforms of cars operated by them, except the open cars having running boards along the sides and having seats accessible directly from such running boards. On such cars smoking will be permitted on the four rear seats of each car including the seats on the back platform. The order also prohibits smoking or carrying lighted cigars, etc., in the stations, station platforms, station stairways, waiting rooms, waiting cars or shelters. The companies are required to enforce these regulations and to post conspicuously in cars and stations, proper notices stating that such practices are prohibited.

The Manual-Control Block-Signal System of the New York, New Haven & Hartford R.R., an old "lock and block" system, is to be replaced by an extension of the electric automatic system recently put into trial use between Stamford and South Norwalk. The old "banjo" shape signals will be replaced by three-position semaphores operating in the upper left-hand quadrant. The signals are to be displayed below cross bridges and to the right of tracks governed. The lighting and operating will be by alternating current furnished from independent signal-current generators in the Cos Cob power house of the New York terminal electric-traction zone. One pair of feeders will be carried at the south of the tracks on the catenary-support bridges and a duplicate pair on the north side. The interlocking towers will have track approach annunciators, approach and route locking with the usual detector locking guards. The home signals at interlocking towers are [illegible]-automatic—controlled from the tower, but thrown to "stop" position automatically by the presence of a train.

Panama Canal Excavation during July, totaled 3,2[illegible],368 cu.yd., a daily average of 124,691 cu.yd. for each of the 26 working days. This is the highest July average on record, and the highest daily average for any month since November, 1910. It closely approaches the high average of the winter of 1908-09, the high-water mark of the canal excavation.

Rainfall during July was 7.28 in. The amount of concrete placed during the month was 17,568 cu.yd. and 191,445 cu.yd. of fill was placed in dams.

Hydraulic Dredging, Culebra Cut, Panama Canal—A relay pumping station is being installed on the west side of Culebra cut, near the bridge of the Panama R.R. over the Rio Grande, in anticipation of dredge excavation at the foot of Cucaracha slide. The plan as at present contemplated involves passing the discharge pipe-line of two suction dredges across the cut on pontoons, and up the west bank to the relay station, thence down the valley of the Rio Grande about 4000 ft. to discharge west of Cerro Luisa, the hill at the outer end of Pedro Miguel dam. It is probable that an earth dam will be thrown from the lower end of Pedro Miguel lock to the hills on the other side of the river, a distance of about 1500 ft., and the swamp west of the lock will be filled with spoil from the slide. ["Canal Record," June 11, 1913.]

The Final Filling of Gatun Lake to the operating level 85 ft. above sea level is now in progress. The "Canal Record" of July 2 states that the last of the three 8x18-ft. sluices through the spillway of the Gatun dam was closed on June 27, with the surface of the lake 48.25 ft. above sea level. It is not expected to open the gates again, and the lake may be considered to be on its final rise to operating level. Under conditions of normal runoff, as determined by measurements recorded over the watershed for the past 23 years, the lake should reach 85 ft. about Dec. 1, 1913; it should rise to 71 ft., the elevation of the railroad at Gorgona, about the first of October. At that height it would afford a 21-ft. channel. The top of the dike at Gamboa, protecting Culebra Cut from flooding through its north end, is 72.2 ft. above sea level, but it is anticipated that the water will be let into the Cut before the lake has reached that elevation, which it may do about the first of November.

Thirty Million Dollars for Inter-Coastal Canals along the Atlantic Coast is asked for by the War Department, acting on the recommendation of Brig.-Gen. Bixby, recently retired as Chief of Engineers, United States Army. The recommendations of Gen. Bixby are based on the report of the special board of five army engineers, which for several years, under the direction of Congress, has been surveying the feasibility and advisability of constructing a chain of inland waterways from Boston, Mass., to Beaufort, N. C. A preliminary report of that board was submitted to Congress last year; as a result, Congress has already appropriated funds for the purchase of the Albemarle and Chesapeake Canal, connecting Norfolk, Va., to the North Carolina Sounds, and giving a straight inside route beyond Cape Hatteras. The new recommendations are as follows:

(1) The purchase of the Chesapeake and Delaware Canal, connecting the Chesapeake and Delaware Bays, at a cost not exceeding $2,514,290.

(2) The expenditure of approximately $8,000,000 by the Government in transforming this canal from a lock canal into a sea-level canal, with a depth of 12 ft. throughout and a 90-ft. width at the bottom.

(3) The postponement of the ultimate deepening of this canal to 25 ft. until after the effect of the new 12-ft. canal upon commerce is apparent.

(4) The construction of a lock canal 12 ft. deep and 90 ft. wide from the Delaware River to New York Harbor, at a total cost of $20,000,000, so built as to provide for economical enlargement of the canal to 25-ft. depth and 125 ft. of width, and the future reduction of the canal to a sea-level basis.

(5) The ultimate development of this canal into a 25-ft. deep sea-lvel canal, in cooperation with New Jersey and other benefited states.

The officers who made the survey, recommended a 25-ft. canal, connecting Delaware Bay with New York Harbor, but Gen. Bixby, approving of the Board of Engineers of Rivers and Harbors, advises now the construction of a 12-ft. canal, leaving a 25-ft. depth for future work.

The City Manager Plan continues to make progress. The commissioners of Little Falls, Minn., have decided for it as an adjunct of the commission plan. The commissioners themselves will receive salaries of $100 a year. The charter com-

mission of Youngstown, Ohio, is committed to the city manager plan; the like commission of Elyria, Ohio, has decided to submit the plan to the voters; and at Dayton, Ohio, the city manager ticket recently won in the charter commission election. In Dayton, John H. Patterson, of the National Cash Register Co. and leader during the recent floods, was the principal advocate of the city manager and commission plans and received 15,000 of the 18,000 votes cast at the charter commission election. We are indebted to H. S. Gilbertson, assistant secretary of the National Short Ballot Organization, for the foregoing information.

An Experimental Sewage-Disposal Station is Under construction at Indianapolis, Ind. It is hoped that the station will be in operation by the middle of September. The station is being built under the direction of H. W. Klausmann, City Civil Engineer of Indianapolis.

Sewage-Treatment Works for Syracuse, N. Y., will probably be built after the intercepting sewers now under construction are completed. Apparently no specific date has been set for beginning the construction of the sewage-treatment works, although we understand the subject is undergoing investigation. Glenn D. Holmes is chief engineer of the Syracuse Intercepting Sewer Board, and Henry C. Allen is City Engineer of Syracuse.

Water-Supply Problems at Denver, Colo., connected with the proposed purchase or construction of works by the city are to be investigated by an engineering board of three members, appointed by the local Public Utilities Commission. The three engineers are: A. Lincoln Fellows and C. P. Allen, of Denver, and E. C. Van Vlest, of Colorado Springs. They are to examine and report on all propositions submitted to the city by the Denver Union Water Co., or others, and are also to estimate the cost of purchasing or building works. They are also to report on the feasibility and cost of diverting a water-supply from the western slope.

A Good Roads Exhibit will be held at the Ohio State Fair, Columbus, Sept. 1-5. The exhibit will occupy the whole of the building devoted heretofore to manufactures. In connection with the exhibit daily trips will be made to experimental roads in the vicinity. The exhibit is in charge of James R. Marker, State Highway Commissioner of Ohio, whose office is at Columbus.

Lumber Statistics for 1911—According to three recent bulletins (Forest Products, Nos. 2, 8 and 9), of the U. S. Bureau of Census, the total number of sawmills in operation in 1911, was 28,107, which was a smaller number than any year since 1907. The total output was approximately 37 billion feet, b.m., or less than any previous year since 1906, with one exception in 1908. Exports of lumber products amounted to nearly 3 billion feet, b.m., valued at over $60,000,000, a considerable increase over the lumber exports of 1910. Nearly a billion feet, b.m., of lumber, including logs and unsawed cabinet woods, valued at over $21,000,000, were imported; nearly nine-tenths of the imports came from Canada.

Estimates by the Bureau of Corporations place the total stand of merchantable timber left in the United States at 2800 billion feet, b.m. Between 1.3 and 1.85% of this amount is being annually cut for lumber alone. Making no allowance for growth, decay or fire, the present stand of timber suitable for lumber, if used for this purpose only, will not last 60 years, at the present rate of consumption. The U. S. Forest Service estimates the present average forest growth at about one-third the present rate of consumption of timber for all purposes. The states producing the largest outputs are Washington, Louisiana and Mississippi respectively.

Lumber is cut from a hundred or more species, but the bulk of the product comes from less than a dozen species. Four species of Southern yellow pine, Douglas fir, and two species of white pine supply more than half the lumber of the United States. Western woods are slowly increasing in relative importance, while in general Eastern woods are remaining stationary or declining.

The number of crossties purchased by the steam and electric railways of the United States in 1911 was over 135,000,000, being 9% less than the number of the previous year. Only 8.2% of the total were for new track—the lowest percentage since 1908. In 1911 over 73,000,000 gal. of creosote were used by wood-preserving plants; 70% of this oil is imported from England and Germany. Crossties formed 74% of all the material given a preservative treatment. The number of ties treated by steam railways after purchase has increased greatly, showing the tendency of steam railways to build and operate their own plants. Over half of the treated ties were treated with creosote, 27.8% with a solution of zinc chloride, and the remainder with combinations of these preservatives, crude oil and various other materials.

The number of poles purchased in 1911 was 2,418,000, or nearly 400,000 less than in 1910. The decrease in purchases of 1911 compared to 1910 was confined to telephone and telegraph companies and steam railways, while substantial increases in purchases were reported by electric railway, light and power companies. Over 90% of the poles were either cedar, chestnut, oak, pine or cypress. About 19% of the total received some kind of preservative treatment.

PERSONALS

Mr. E. W. Peabody has been appointed Statistician of the Missouri, Kansas & Texas Ry., with headquarters at St. Louis, Mo.

Mr. Robert H. Pearson, recently Superintendent of the inspection department of the Globe Indemnity Co., New York City, has been promoted to be Consulting Engineer.

Mr. J. E. McQuillen, recently Master Mechanic of the Gulf, Colorado & Santa Fé Ry., at Silsbee, Tex., has been promoted to be Mechanical Superintendent at Cleburne, Tex.

Mr. C. S. Muzzarelli has resigned as Consulting Engineer of the Dover Boiler Works, New York City, to become Vice-President in charge of the construction department of the New York Structural Co.

Mr. Edward J. McGuire, of Buffalo, N. Y., has been appointed Engineer of the division of light, heat and power of the Public Service Commission of New York, Second District, at a salary of $5000 per annum.

Mr. Seymour W. Cheney, formerly General Superintendent of the La Crosse Gas & Electric Co., La Crosse, Wis., has been appointed Assistant Chief Engineer of the American Public Utilities Co., Grand Rapids, Mich.

Mr. W. H. Farrell, Superintendent of Freight Service of the Grand Trunk Ry., Montreal, Que., has been appointed Superintendent of the Toronto, Ont., terminals, succeeding Mr. G. A. Stokes, assigned to other duties.

Mr. Charles L. Rogers, President and General Manager of the Sligo Furnace Co. and the Sligo & Eastern R.R., St. Louis, Mo., has been appointed Representative of the American Car & Foundry Co., at London, England.

Mr. H. B. Scharnberg, formerly with the Lackawanna & Wyoming Valley R.R., Scranton, Penn., has been appointed Chief Engineer of power stations of the Rio de Janeiro Tramway, Light & Power Co., Rio de Janeiro, Brazil.

Mr. T. J. Fleming, Assoc. Am. Inst. E. E., recently Representative of the Westinghouse Electric & Manufacturing Co., of Pittsburgh, Penn., in Buenos Ayres, Argentina, has been appointed Manager of the Red Wing, Minn., Gas, Light & Power Co.

Mr. Byron T. Gifford, M. Am. Soc. M. E., recently Manager of the general engineering department of the American District Steam Co., has been appointed Chief Engineer of the American Public Utilities Co., Grand Rapids, Mich., succeeding Mr. Samuel C. Schaffner.

Mr. Henry W. Crowell, M. Am. Soc. M. E., recently Assistant Superintendent of Inspection of the Fidelity & Casualty Co., of New York City, has been appointed Superintendent of the inspection department of the Globe Indemnity Co., New York City, succeeding Mr. Robert H. Pearson, promoted, as noted elsewhere.

Lieut.-Col. David Du Bois Gaillard, Corps of Engineers, U. S. A., a member of the Isthmian Canal Commission and Division Engineer of the Central division of the Panama Canal, has returned to this country suffering from a severe nervous breakdown, due chiefly to his arduous work of six years at Panama. Lieut.-Col Gaillard is now at the Henry Phipps Clinic, in Baltimore, Md., where he will receive all possible assistance in recovering his health.

Mr. James A. Lowell, of Newton, Mass., has been appointed Chairman of the newly created Board of Labor and Industries of Massachusetts. The other members are Messrs. James W. Crook, Professor of economics at Amherst College; Channing Smith, a woolen manufacturer of Leicester; William Acton, of Fall River; and Mrs. David R. Dewey, of Cambridge. This board will have supervision of the enforcement of labor laws. The salary of the Chairman is $1500 per annum and the other members $1000.

Col. William C. Gorgas, Assistant Surgeon General, U. S. A., Chief Sanitary Officer of the Panama Canal Zone, has accepted

the invitation sent to him by the Chamber of Mines, Johannesburg, South Africa, to undertake the improvement of sanitary conditions on the Rand, according to a newspaper dispatch from Johannesburg, dated Aug. 14. The appointment is subject to the approval of the United States Government, and will probably not take effect until the completion of the Panama Canal work.

Mr. Charles E. Bright, Assoc. M. Am. Soc. C. E., United States Superintendent, Colbert Shoals Canal, Riverton, Ala., has been granted a furlough in order that he may accept the position of Engineer in charge of the repairs to the city reservoir at Nashville, Tenn., to which office he was recently appointed by the Nashville Board of Public Works. The failure of one of the walls of the city reservoir was described and illustrated in our issue of Nov. 14, 1912. The method by which the repair work is to be accomplished was described in detail in an abstract of the report of Mr. Rudolph Hering, M. Am. Soc. C. E., Consulting Engineer, of New York City, in our issue of Feb. 6, 1913.

Mr. Nathaniel Ewing, former Chairman of the Pennsylvania State Railroad Commission, has been appointed Chairman of the new Public Service Commission of Pennsylvania, at a salary of $10,500 per annum. Other members of the commission are Ex-Governor Samuel W. Pennypacker and Messrs. Milton J. Brecht, both former members of the Railroad Commission; Emory R. Johnson, Professor of transportation at the University of Pennsylvania; Charles F. Wright, of Susquehanna; Frank M. Wallace, of Erie; S. L. R. Tone, M. Am. Soc. C. E., of Alleghany, Second Vice-President of the Pittsburgh Rys. The salary of the members is $10,000 per annum.

Mr. Samuel McC. Young, M. Am. Soc. C. E., of Guerin & Young, New Orleans, La., has been appointed Chief Engineer of the Board of Commissioners of the Port of New Orleans, succeeding Mr. A. C. Bell, resigned, as noted in our issue of Aug. 7. Mr. Young is a graduate of Washington and Lee University. For several years he was on the engineering staff of the Choctaw, Oklahoma & Gulf R.R., advancing from rodman to Resident Engineer. During 1904 and 1905 he was a contractor for the construction of several water-works and electric power plants in Mississippi and Louisiana. Later he was Locating Engineer and Division Engineer of the New Orleans Great Northern R.R. For a few years following 1909 he was associated with Mr. J. F. Coleman, M. Am. Soc. C. E., Consulting Engineer, of New Orleans. Previous to August, 1912, when the partnership of Guerin & Young was founded, Mr. Young had been Engineer and Superintendent of Construction of the Rome Ry. & Light Co., Rome, Ga.

OBITUARY

William R. McClellan, an Assistant City Engineer, of Tacoma, Wash., was killed in an automobile accident, July 26.

John V. Smith, a former President of the Alabama State Railroad Commission, died recently.

Charles M. McCarthy, President of the Pacific Construction Co., of San Francisco, Calif., died recently at his home in Alameda, Calif.

Arthur Davis Berstler, Assistant Engineer, Board of Water Supply, City of New York, died at the home of his parents in Millburn, N. J., Aug. 12. He was 30 years old.

Joseph J. Coughlin, Roadmaster of the Las Vegas & Tonopah R.R., Las Vegas, Nev., was killed while on an inspection trip by falling from a gasoline-motor inspection car, Aug. 10.

Charles G. Jamer, Supervising Engineer of the Department of Correction, New York City, died at his home in Brooklyn, Aug. 14. He was in his 47th year and had been a resident of Brooklyn all his life.

John Calvin Locke, a civil engineer with the Topographical Bureau of Brooklyn, New York City, died at the Methodist Episcopal Hospital, in Brooklyn, Aug. 15, aged 44 years. He attended the Massachusetts Institute of Technology from 1890-'94.

J. Merton Taylor, President of the Taylor Instrument Cos., of Rochester, N. Y., died July [illegible]. He was born in Rochester 57 years ago, the son of the late George Taylor, founder of the Taylor Instrument Cos., manufacturers of thermometers, barometers and other scientific instruments. Mr. Taylor became President of the companies in 1910. He is survived by a widow and two daughters.

Ned Herbert Janvrin, M. Am. Soc. C. E., Assistant Engineer, Board of Water Supply of the City of New York, died July 16, aged 42 years. He was born at Somerville, Mass., and graduated from the Massachusetts Institute of Technology in 1894. For the first three years following his graduation he was successively with the Boston Bridge Works, the United States Geological Survey and with Mr. J. R. Worcester, M. Am. Soc. C. E., Consulting Engineer, of Boston, Mass. In 1897 he accepted a position with the Pennsylvania Steel Co. in structural-design work. From here he went to Kansas City, Mo., where for a time he was Assistant Bridge Engineer of the Metropolitan Street Ry., later he was employed by Waddell & Hedrick, Consulting Engineers. In 1900 Mr. Janvrin returned to Boston, where he was employed by Mr. J. R. Worcester on structural steel work. In April, 1901, he accepted a position with the American Bridge Co., where he remained until 1905, two years of which he was Resident Engineer in charge of erection of all work in the vicinity of Louisville, Ky., and Cincinnati, Ohio. For the past seven years he had been with the New York City Board of Water Supply.

ENGINEERING SOCIETIES

COMING MEETINGS

INTERNATIONAL ASSOCIATION OF MUNICIPAL ELECTRICIANS.
Aug. 19-22. Annual Convention at Watertown, N. Y. Secy., Clarence R. George, Houston, Tex.

AMERICAN BOILER MANUFACTURERS' ASSOCIATION.
Sept. 1-4. Annual convention at Cleveland, Ohio. Secy., J. D. Farasey, 37th St., Cleveland, Ohio.

INTERNATIONAL ASSOCIATION OF FIRE ENGINEERS.
Sept. 1-6. Annual convention at New York City. Secy., Jas. McFall, Roanoke, Va.

ROADMASTERS' AND MAINTENANCE OF WAY ASSOCIATION.
Sept. 8-12. Annual convention at Chicago, Ill. Secy., L. C. Ryan, Chicago and North Western Ry., Stirling, Ill.

AMERICAN ELECTROCHEMICAL SOCIETY.
Sept. 9-11. The 24th general meeting at Denver. Secy., Prof. Jos. W. Richards, Lehigh University, South Bethlehem, Penn.

NEW ENGLAND WATER-WORKS ASSOCIATION.
Sept. 10-12. Annual convention at Philadelphia, Penn. Secy., Willard Kent, Narragansett Pier, R. I.

AMERICAN PUBLIC HEALTH ASSOCIATION.
Sept. 9-13. Annual meeting at Colorado Springs, Colo. Secy., S. M. Guma, 755 Boylston St., Boston.

MASTER CAR AND LOCOMOTIVE PAINTERS' ASSOCIATION.
Sept. 9-12. Annual convention at Ottawa, Ont. Secy., A. P. Dane, Reading, Mass.

NATIONAL PAVING BRICK MANUFACTURERS ASSOCIATION.
Sept. 17-18. Tenth annual meeting at Cleveland, Ohio. Secy., W. P. Blair, 824 Engineers' Building, Cleveland, Ohio.

ILLUMINATING ENGINEERING SOCIETY.
Sept. 22-26. Annual Convention at Pittsburgh, Penn. Secy., I. D. Israel, 29 W. 39th St., New York City.

AMERICAN MINE SAFETY ASSOCIATION.
Sept. 22-24. Annual meeting at Pittsburgh, Penn. Chairman, H. M. Wilson, 40th and Butler Sts., Pittsburgh, Penn.

AMERICAN ROAD CONGRESS.
Sept. 29-Oct. 5. Annual convention at Detroit, Mich.

AMERICAN SOCIETY OF MUNICIPAL IMPROVEMENTS.
Oct. 7-10. Annual convention at Wilmington, Del. Secy., Geo. H. McGovern, Chamber of Commerce, Wilmington, Del.

RAILWAY SIGNAL ASSOCIATION.
Oct. 14-16. Annual meeting at Nashville, Tenn. Secy., C. C. Rosenberg, Times Building, Bethlehem, Penn.

AMERICAN ELECTRIC RAILWAY ASSOCIATION.
Oct. 13-17. Annual convention at Atlantic City, N. J. Secy., H. C. Donecker, 29 West 39th St., New York City.

AMERICAN GAS INSTITUTE.
Oct. 15-17. Annual meeting at Richmond, Va. Secy., Geo. G. Ramsdell, 29 West 39th St., New York City.

AMERICAN MINING CONGRESS.
Oct. 20-25. Annual convention at Philadelphia, Penn. Secy., J. Callbreath, Majestic Building, Denver, Colo.

AMERICAN RAILWAY BRIDGE AND BUILDING ASSOCIATION.
Oct. 21-23. Annual convention at Montreal, Que. Secy., C. A. Lichty, 319 North Waller Ave., Chicago, Ill.

International Association for the Prevention of Smoke—The eighth annual convention will be held at Pittsburgh, Penn., Sept. 9-12. The headquarters will be at the Fort Pitt Hotel. Papers will be read on the subject of the "Abatement of Locomotive Smoke," and upon the "Standardization of the Installation of Boilers and Stokers." For further information address the Vice-President, J. M. Searle, 407 Nixon Building, Pittsburgh, Penn.